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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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MARCH 1932-FEBRUARY 1933

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ABBREVIATIONS

a.—article(s), report(s)
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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 1 ▽ MARCH, 1932

ON THE COLUMBIA RIVER HIGHWAY IN OREGON

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PUBLIC ROADS

▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 1

March, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE WESTERN STATES TRAFFIC SURVEY

Reported by L. E. PEABODY, Senior Highway Economist, U. S. Bureau of Public Roads

THE REPORT of a traffic survey of the Federal-aid highway system in the Western States has recently been published by the Bureau of Public Roads.[1] This survey was conducted by the bureau in cooperation with the State highway departments of Arizona, California, Colorado, Idaho, Nebraska, New Mexico, Nevada, Oregon, Utah, Washington, and Wyoming. The report gives the essential facts regarding the present density, types, capacities, and distribution of traffic units and forecasts of future traffic as a basis for planning highway development to serve present and future needs. A summary of the principal facts brought out by the survey is given in the following pages.

On the Ridge Route in California

SUMMARY OF CONCLUSIONS

1. Annual use of the Federal-aid highway system is approximately 8,400,000,000 vehicle-miles. Intensity of use per mile is greatest in California and least in Nevada. Travel is concentrated. For example, on U. S. 91 from Springville to Logan, Utah, comprising 6.6 per cent of the highway mileage of the State, is found more than 40 per cent of the total vehicle mileage in Utah, and more than 40 per cent of the truck mileage. In Nevada 64 miles of the Federal-aid system near Reno, 4.2 per cent of the total in the State, carries 25 per cent of the State's total vehicle mileage and about 17 per cent of the truck mileage.

2. Foreign traffic varies from 4.9 per cent of the total motor traffic in California to 38.5 per cent in Arizona, and averages 15 per cent throughout the whole area. Six of the eleven States have more than 20 per cent, and three States more than 30 per cent of foreign traffic. By foreign traffic is meant travel by vehicles from outside the particular State under discussion.

3. Trucks under 3 tons capacity constitute 85.5 per cent of all truck traffic; more than 50 per cent of all trucks operating in these States are in the 1 to 1½ ton group. Less than 6 per cent are 5-ton or over. California has the heaviest truck traffic, with 23.1 per cent in the group of 3 tons and over; New Mexico has the smallest number of heavy trucks, with 5.2 per cent having capacities of 3 tons and over.

4. Nearly half of all trucks travel under 80 miles per day; 12 per cent travel in excess of 200 miles per day. The average daily travel is 103 miles, the median daily travel 82 miles. The proportion of foreign trucks making exceptionally long trips to total foreign trucks is nearly five times the proportion of local trucks making similar trips. The average travel of foreign trucks is 154 miles, more than one and one-half times the average daily travel of local trucks.

5. Approximately one-half of all trucks are city owned. The remaining half is nearly equally divided among owners living in villages (under 2,500 population) or on farms. In Wyoming, a State with large areas of unproductive land and a small population largely concentrated in villages along the highways and railroads, only 23 per cent of the trucks are city owned, 34 per cent are village owned, and 43 per cent are farm owned. California is at the other extreme with two large urban areas and numerous smaller cities. Here

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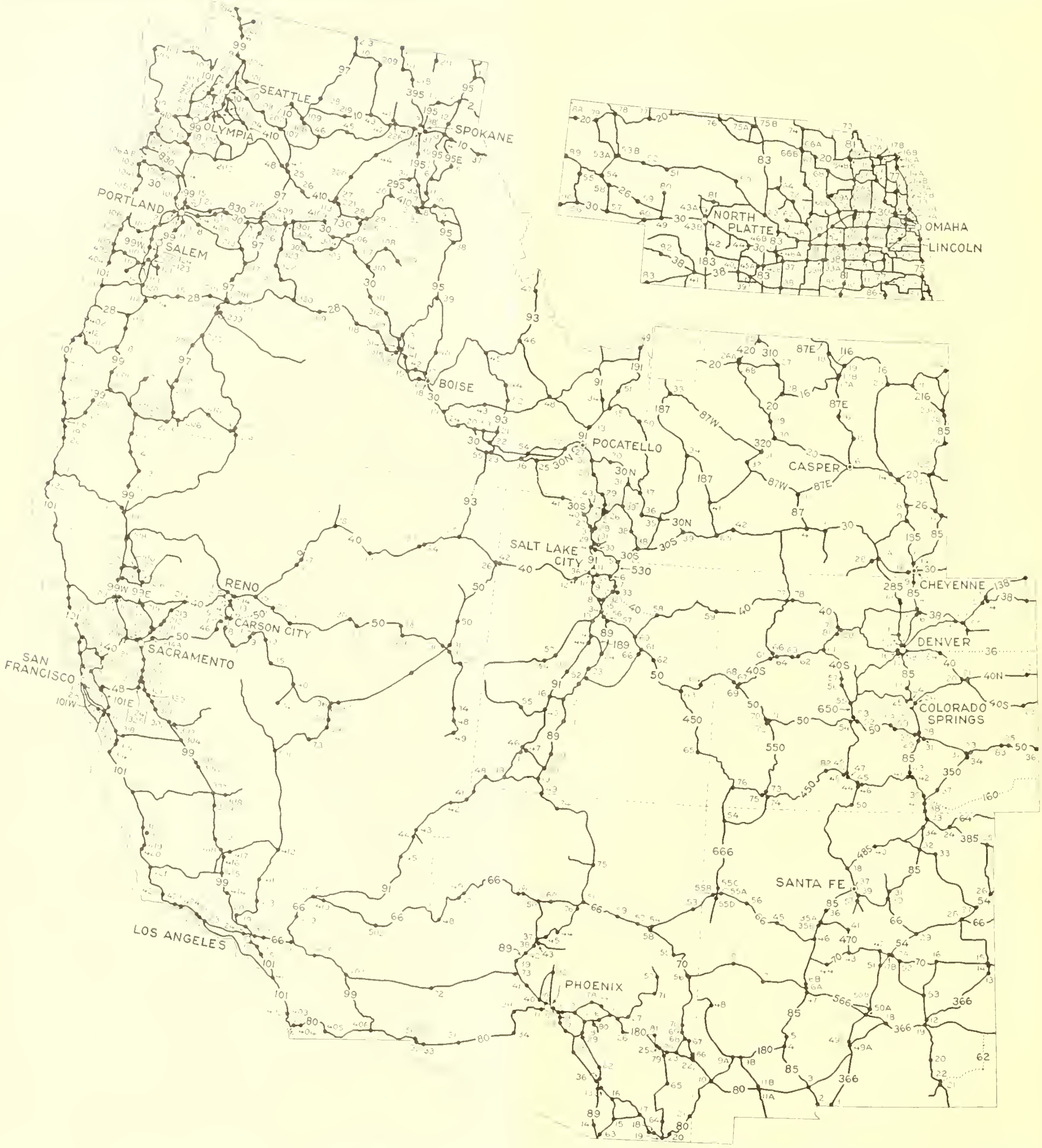


Figure 1.—Highway Map of 11 Western States, Showing Location of Traffic Recording Stations. U. S. Highway Numbers are Shown in Bold Numerals, Numbers of Recording Stations in Light Numerals

city-owned trucks are 66 per cent of the total, and the remainder is divided equally between farms and villages. There is definitely a greater proportionate use of the highways by rural trucks than by urban trucks.

6. The effect of physical barriers upon truck traffic is pronounced. Most foreign trucking originates in adjoining States.

7. Eighty-six per cent of all trucks are operated by their owners; contract haulers operate 8.7 per cent, and common carriers operate 5.5 per cent of all trucks. The proportion of owner-operated trucks is six times that of commercial trucks. Common-carrier trucks constitute less than 1½ per cent of all motor traffic. Approximately 80 per cent of common carriers are engaged in intrastate hauling. Variation in demand for common-carrier service among the States is dependent mainly upon the variations in density and distribution of population within the area, and upon the amount of registration fee charged by the State. A comparison of common-carrier traffic with the num-

ber of such carriers registered indicates an intensity of usage of common carriers about four times that of all trucks.

8. Approximately 57 per cent of all passenger cars travel less than 140 miles per day; 4 per cent travel 400 miles or more per day. The average daily mileage of passenger cars is nearly 50 per cent greater than the average daily mileage of trucks. The average daily travel of foreign passenger cars is nearly double that of local passenger cars. Ten per cent of foreign passenger cars travel over 400 miles per day.

9. In but three States, Nebraska, Nevada, and Wyoming, does the proportion of city-owned passenger cars fall below 50 per cent; in California the proportion is 73.2 per cent. In the whole area city-owned passenger cars constitute 61 per cent. A comparison of truck and passenger car situs of ownership indicates that in every State the proportion of city-owned cars is much higher than that of city-owned trucks. City-owned cars travel an average of 169 miles per day; village-owned cars, 129 miles per day; farm-owned cars, 99 miles per day.

10. Passenger-car traffic is less affected than truck traffic by difficult topography or climate. More than one-seventh of the foreign passenger-car traffic in the Western States originates east of the Mississippi River.

11. The number of passengers per car averages 2.42, with slight variation in the average among individual States. Nearly two-thirds of all passenger cars carry either one or two passengers.

12. The Federal-aid highways of the 11 States were classified upon the basis of present traffic and traffic forecasts for 1935 and 1940. Routes or sections of routes carrying 1,500 or more motor vehicles per day are classed as major routes, those carrying 600 to 1,500 per day as intermediate routes, and those carrying less than 600 vehicles daily are classed as minor routes. The routes or sections of routes are grouped in this way on the basis of observed 1930 traffic, and the estimated traffic for 1935 and 1940 is employed in a similar manner to indicate the probable classification in those years.

Slightly over 11 per cent of the Federal-aid highways carried over 1,500 motor vehicles per day in 1930. Nearly one-half of the mileage classed as major lies in the State of California, about one-fifth is located in Washington, and the remainder is scattered mainly in short sections throughout the other States.

Eighteen per cent of the mileage carried between 600 and 1,500 motor vehicles per day in 1930, about one-quarter of which was in the State of Nebraska, with nearly as much mileage of intermediate classification in California.

The remaining 66 per cent of the classified mileage carried less than 600 motor vehicles per day in 1930. Five per cent of the Federal-aid system could not be classified.

In 1935 the respective mileage classifications are expected to be as follows: major 14 per cent; intermediate, 22 per cent; minor, 59 per cent. The expected classification for 1940 is as follows: major, 16 per cent; intermediate, 25 per cent; minor 54 per cent.

THE METHODS OF THE SURVEY DESCRIBED

The traffic survey was begun in September, 1929, in all States except Washington and Nevada, where field operations were instituted in October and November, respectively. Traffic data was recorded at 899 points in the 11 States with locations as shown in Figure 1.

In addition to counting the volume of traffic in each classification of vehicles and upon each route, field men stopped each motorist and distributed franked cards which were returned to the Bureau of Public Roads. By means of these cards information was obtained regarding the day's mileage traveled, the origin and destination of the vehicle, the number of passengers carried, the capacity of trucks, the situs of ownership (farm, village, or city) and the State of registration. During the last three months of field operations traffic recorders questioned drivers of trucks, obtained the names of owners, classified them as owner-operators, contract haulers or common carriers, and determined whether operations were interstate or intrastate. Busses were classified simply as interstate or intrastate.

DETAILED TRAFFIC DATA

The average daily traffic upon the highways in the Western States is presented graphically in Figure 2. In the Pacific Coast States is found the largest mileage of highway carrying over 1,500 motor vehicles per day, totaling more than 3,000 miles. The traffic on U. S. Highway 99 is continuously above 1,500 vehicles per day from Indio to Sacramento and from Eugene, Oreg., to Ferndale, Wash., throughout more than two-thirds of its entire length. Between Los Angeles and Bakersfield there is an average of 300 trucks per day. The transportation of oil-well supplies to the oil fields near Bakersfield, and of cotton, hay, and milk to Los Angeles make this an extremely important section from the standpoint of weight of traffic. In addition to the long-haul truck traffic on U. S. 99 between northern and southern California, there is a great deal of local trucking between the cities of the densely populated San Joaquin Valley. From Bakersfield north to Stockton the average daily volume of trucks varies from 400 immediately north of Bakersfield to more than 1,000 trucks per day at Fresno. The average daily travel of trucks on this route is 140 miles and the median daily travel 120 miles, considerably higher than the average and median for all trucks in California.

Approximately one-half the mileage of U. S. 101, between San Diego and Healdsburg, Calif., and a few short sections near cities, carries 1,500 or more motor vehicles per day. The greatest volume of truck traffic is found south of Los Angeles between that city and Whittier, a total of nearly 1,400 trucks per day passing over this section, of which more than 350 are trucks of greater than 3-ton capacity. The greater portion of this traffic is local, as the number of trucks decreases rapidly south of Whittier, but the average between Whittier and San Diego is about 400 trucks per day, and the number of heavy trucks is but slightly less than 100 per day. The average and median daily mileage traveled by trucks on this route between Los Angeles and San Francisco are 120 and 100 miles, respectively.

Colorado has more than 300 miles of highway with a density of 1,500 or more vehicles daily, the mileage being distributed over sections of several routes—U. S. 85, Pueblo to Greeley; U. S. 50, La Junta to Pueblo; and U. S. 285, Denver to Fort Collins. These sections form a continuous route with traffic of this volume from La Junta to Greeley via Denver, and from Denver to Fort Collins.

With the exception of a section from Provo to Brigham, Utah, 104 miles in length, and a few short sections near the cities of Arizona, Idaho, Nebraska, Nevada, and New Mexico, the remainder of the mileage carries less than 1,500 motor vehicles per day.

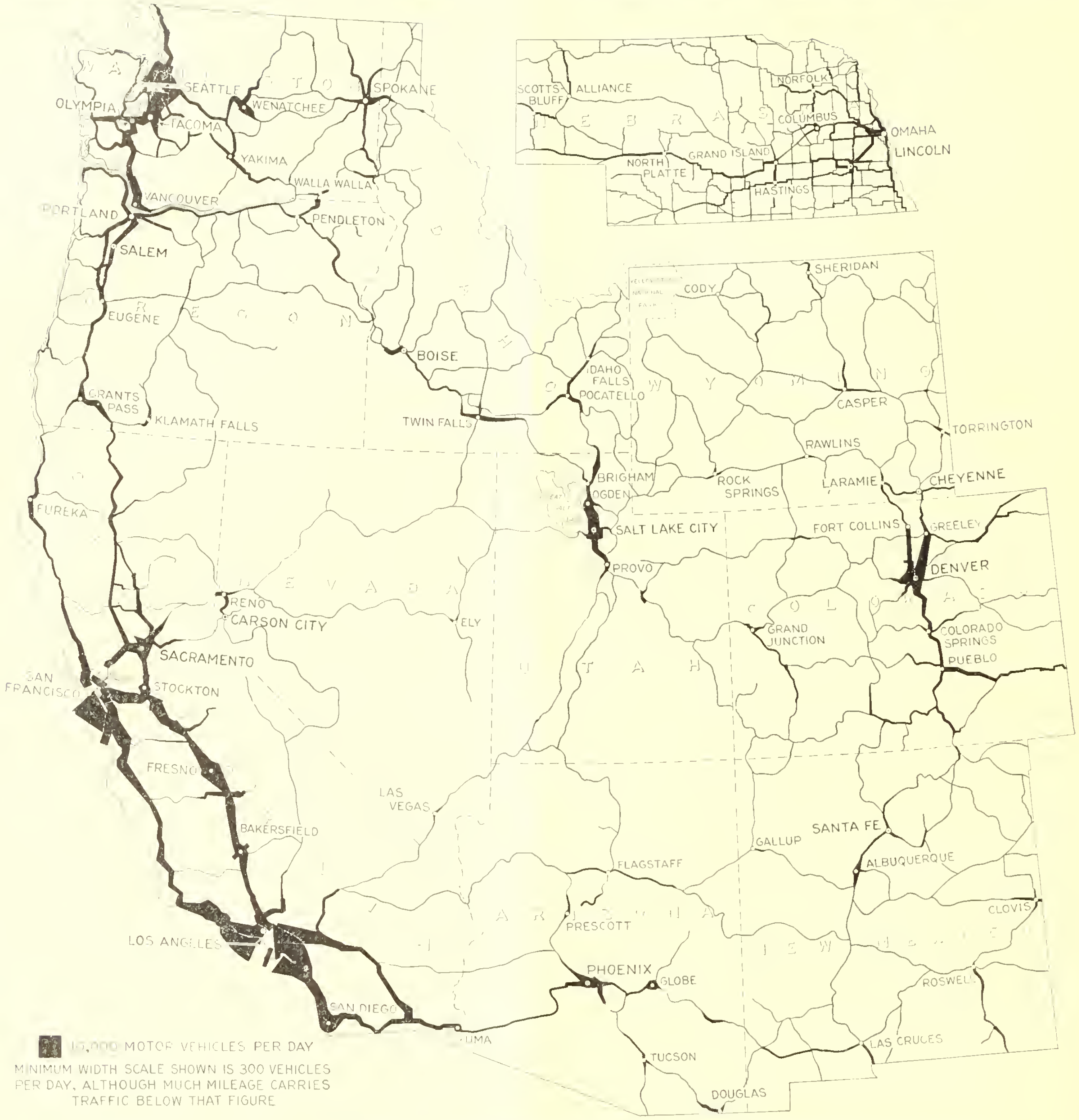


FIGURE 2.—AVERAGE DAILY DENSITY OF TRAFFIC ON THE HIGHWAYS OF THE WESTERN STATES, 1929–1930

FOREIGN TRAFFIC

The percentage of foreign traffic within each State is given in Table 1. These percentages are the ratio of foreign vehicle-miles to total vehicle-miles. It should be noted that a small percentage does not necessarily indicate a small volume of foreign traffic.

The average daily flow of foreign traffic for all States is shown graphically in Figure 3. A comparison of the traffic-flow map (fig. 2) with the foreign traffic-flow map (fig. 3) reveals some interesting contrasts. On U. S. 80 at the California-Arizona line, the volume of foreign traffic on the Arizona side is half again as large as that upon the California side of the border, because of the

TABLE 1.—*Average daily foreign traffic in Western States, expressed in vehicle-miles and in percentage of average total daily traffic*

State	Daily foreign vehicle-miles	Percentage of total vehicle-miles	State	Daily foreign vehicle-miles	Percentage of total vehicle-miles
Arizona	347, 000	38. 5	New Mexico	405, 000	37. 6
California	412, 000	4. 9	Oregon	434, 000	22. 1
Colorado	331, 000	14. 9	Utah	116, 000	14. 3
Idaho	220, 000	22. 4	Washington	358, 000	11. 4
Nebraska	303, 000	12. 7	Wyoming	239, 000	28. 5
Nevada	90, 000	31. 0			

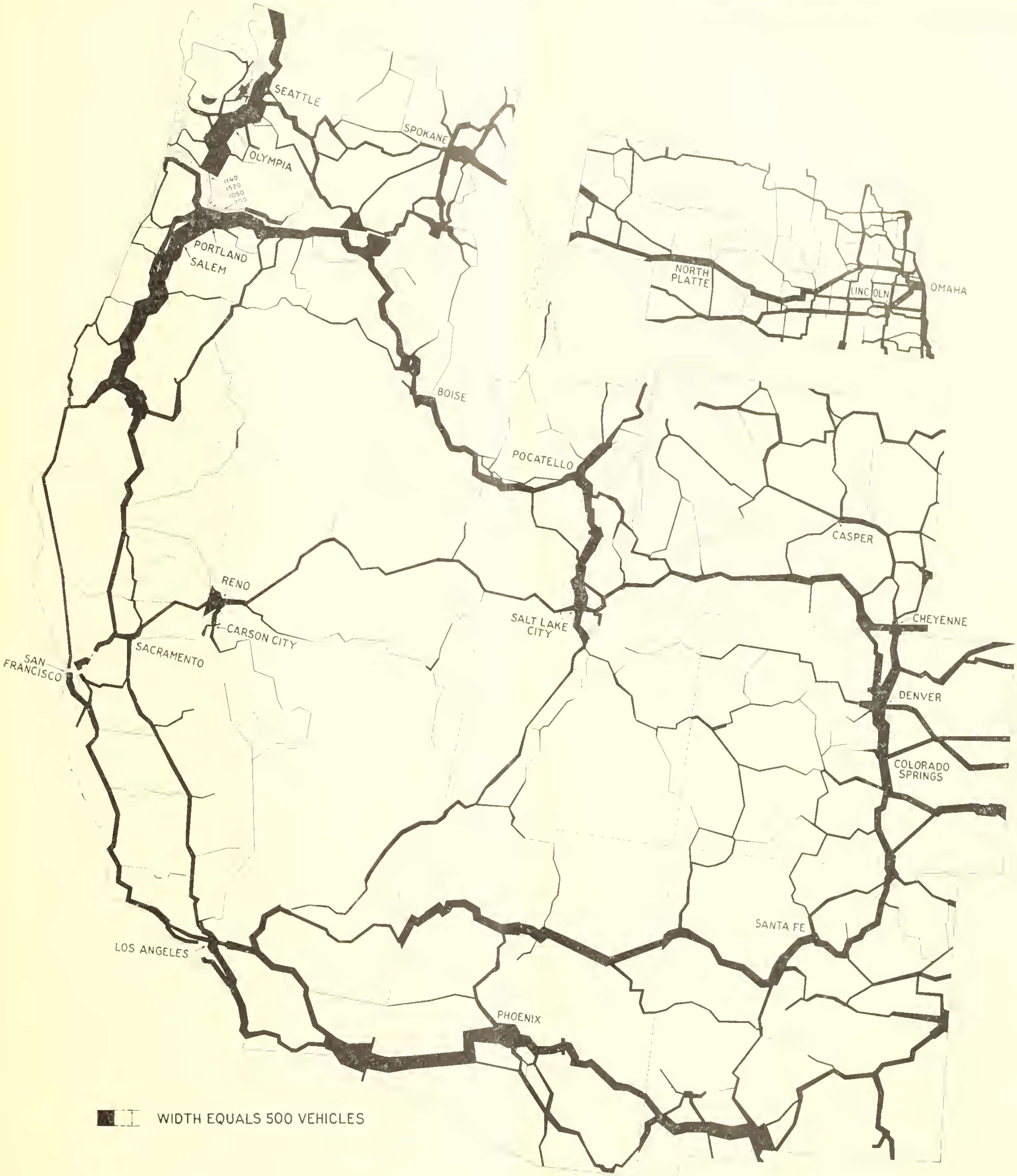


FIGURE 3.—AVERAGE DAILY DENSITY OF FOREIGN TRAFFIC IN THE WESTERN STATES, 1929–1930

large number of cars from the latter State traveling to Yuma. Still more noticeable are the changes in volume of foreign traffic on U. S. 99 as it passes the California-Oregon and Oregon-Washington boundaries. At the California-Oregon line local traffic from California to Ashland and Grants Pass increases the volume of foreign traffic in Oregon to approximately twice that in California, while at the Oregon-Washington line local traffic from Vancouver, Wash., to Portland, Oreg., is responsible for a foreign density in Oregon more than three times that in Washington. This latter difference and those in the immediate vicinity of the Oregon-Idaho border near Weiser and Ontario, Idaho, are not shown upon the map, as the sections are short in length and large in width, and to show them would so distort the map as to blot out other important data.

VARIATIONS IN TRAFFIC DENSITY

Traffic density varies from the averages of the flow maps on any particular day, or during the various seasons of the year and is abnormally high because of fairs, football games, or other sporting events producing exceptional movement. Aside from such abnormal movements, traffic density reaches its normal maximum during the month of August in all States except Arizona, where the month of heaviest travel is March, and in Utah, where September is the month of maximum traffic. A more valuable picture of the variation by seasons of the year, and one which does not accentuate the extremes, is given by the ratio of the average traffic during the six months of heaviest traffic to that during the six months of lightest traffic. The ratios of summer to winter traffic are presented by States for passenger cars and trucks separately in Figures 4 and 5. These data bring out very sharply the effect of climatic differences upon traffic flow; for example, Arizona's passenger-car traffic varies but 7 per cent between winter and summer, while Wyoming's severe winters are reflected in an 89 per cent variation between the two seasons. For truck traffic the figures are somewhat similar, although the comparison of truck traffic at different seasons is seriously affected by the presence or absence of large population centers or industrial areas. Truck traffic is less responsive to seasonal changes. The extreme seasonal variation is best illustrated by Wyoming's passenger-car traffic, which shows an increase of 247 per cent from January to August.

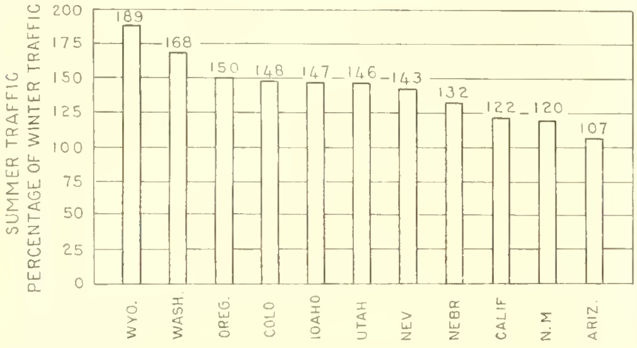


FIGURE 4.—COMPARISON OF SUMMER AND WINTER PASSENGER-CAR TRAFFIC

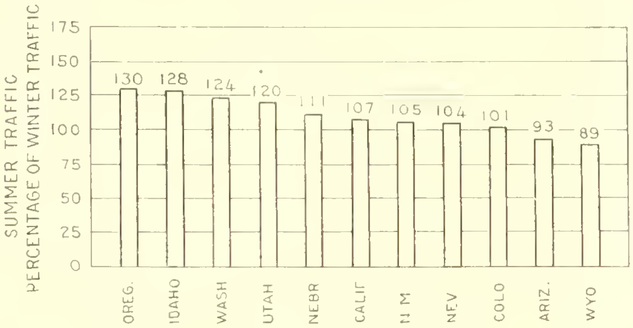


FIGURE 5.—COMPARISON OF SUMMER AND WINTER TRUCK TRAFFIC

U. S. 87, north of Moran, Wyo., illustrates the effect of winter conditions upon traffic flow. Here the average daily density for the winter season is but 8 per cent of that for the entire year. The flow of traffic into Yellowstone Park during the summer months tends to raise the yearly average and depress proportionately the winter average. U. S. 50, south of Glenbrook, Nev., and U. S. 191, north of Trude, Idaho, are other examples. On U. S. 10, north of Virden, Wash., the winter ratio is but 25 per cent of the yearly average, because of winter traffic avoiding Blewett Pass.

In some portions of the southern States of this group the situation is reversed and average winter traffic exceeds the annual average by as much as 26 per cent.

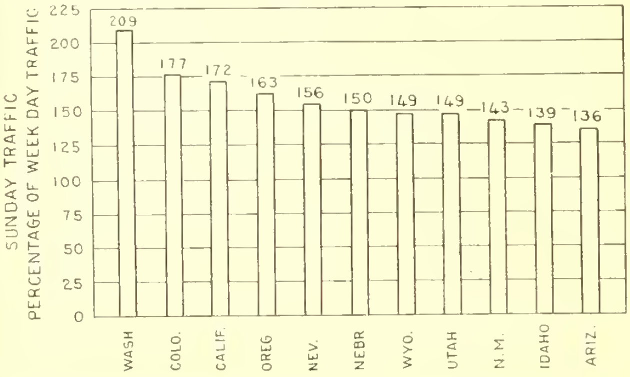


FIGURE 6.—COMPARISON OF SUNDAY AND WEEK-DAY PASSENGER-CAR TRAFFIC

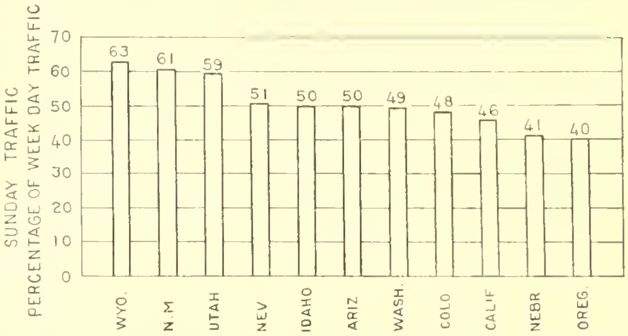


FIGURE 7.—COMPARISON OF SUNDAY AND WEEK-DAY TRUCK TRAFFIC

A high winter average may be due to causes other than travel of winter tourists; for example, on U. S. 180, east of Silver City., N. Mex., truck traffic to copper mines tends to keep the winter average high.

Passenger-car traffic is greater on Sunday than on any other day of the week in all States, while truck traffic is less on Sunday in every State. The ratios of Sunday to average week-day traffic are given for passenger cars and trucks in Figures 6 and 7.

TRUCK TRAFFIC

Highway design and location are dependent not only upon the volume of traffic to be served, but also upon the unit weight of this traffic. In problems which concern the free movement of traffic, such as the width of pavement, the necessity for parallel routes, or the elimination of grade crossings and "bottle necks," density of traffic is the most important factor. In the problems of designing the section or of selecting the most economical pavement, weight of traffic as well as density must be considered. A relatively small highway mileage in each State carries the greatest amount of heavy trucking. Truck traffic upon the highways of the 11 States, including the average daily density of trucks with capacities of 3 tons and over, is presented graphically in Figure 8.

Light trucks are far more numerous than those of any other type on the highways in the Western States. Trucks ranging from 1 to 1½ ton capacity form the most

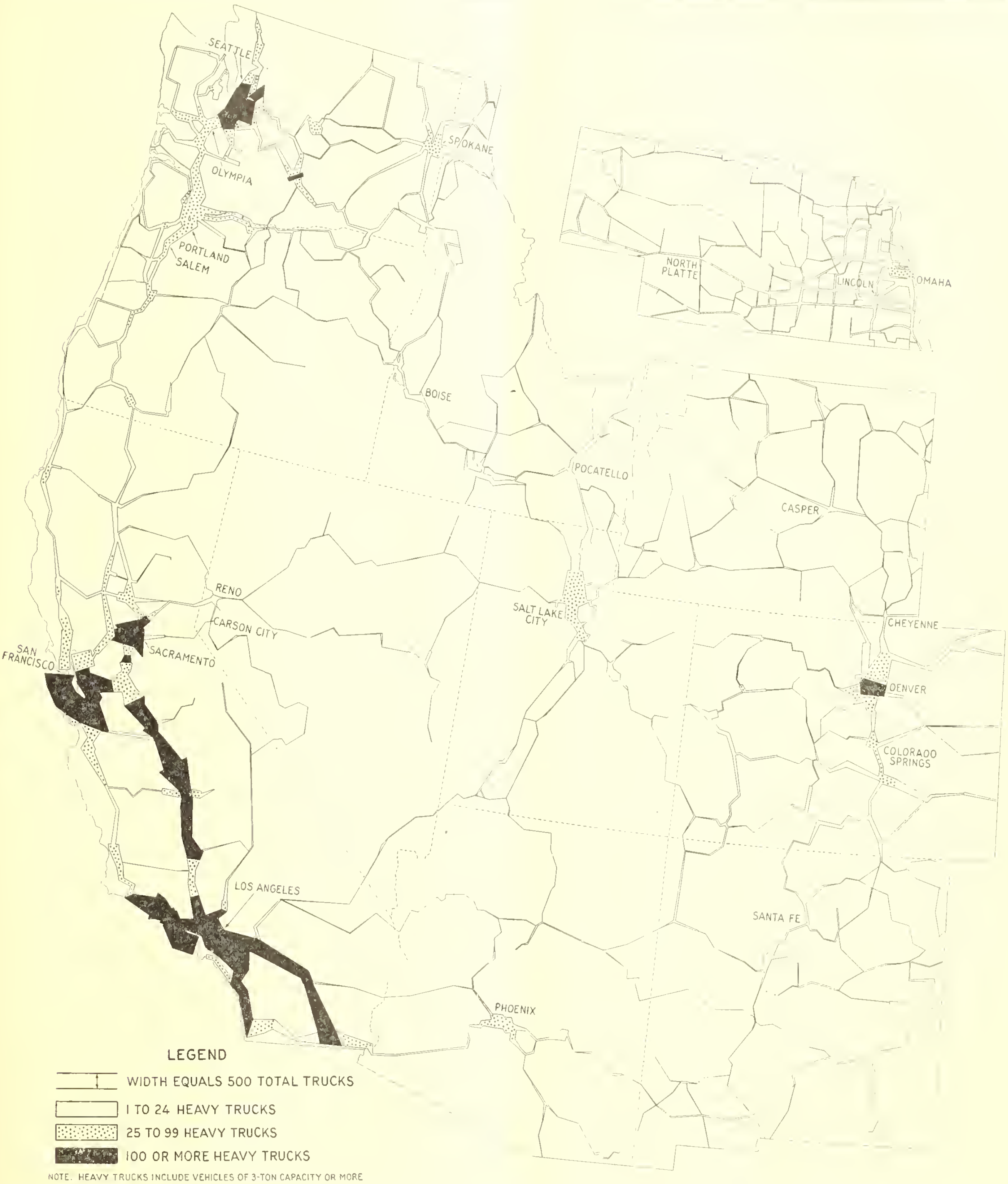


Figure 8.—Average Daily Density of Truck Traffic in the Western States, 1929–1930, Including Density of Heavy Truck Traffic. Width Between Lines Indicates Density of Total Truck Traffic and Symbols Indicate Density of Heavy-Truck Traffic

important group in all States of the survey, and represent more than 50 per cent of all trucks operating in these States. The use of very light delivery trucks and of trucks of slightly greater than 1½-ton capacity is also extensive, trucks of less than 1-ton capacity forming 14.2 per cent and trucks of 2 to 2½ ton capacity forming 17.6 per cent of all trucks. Light trucks of less than 3-ton capacity comprise 85.5 per cent of all trucks. Trucks of greater than 3-ton capacity, while comprising but 14.5 per cent of all trucks, are far more important from the standpoint of highway design. Table 2 gives in detail the relative number of trucks of the various capacities observed on the highways of each State and also the combined percentages for the total number of trucks observed. These percentage distributions are shown graphically in Figure 9. The largest

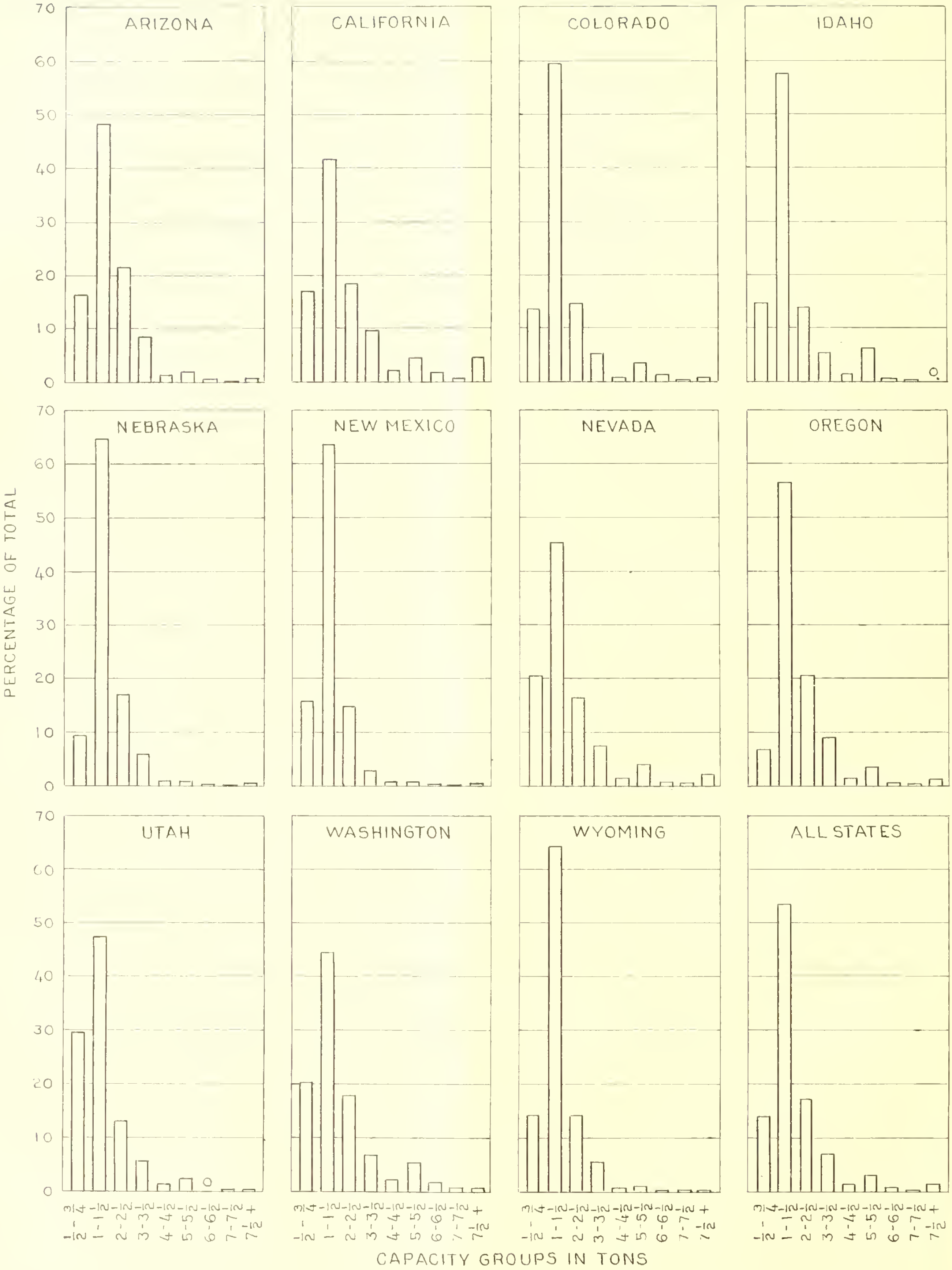
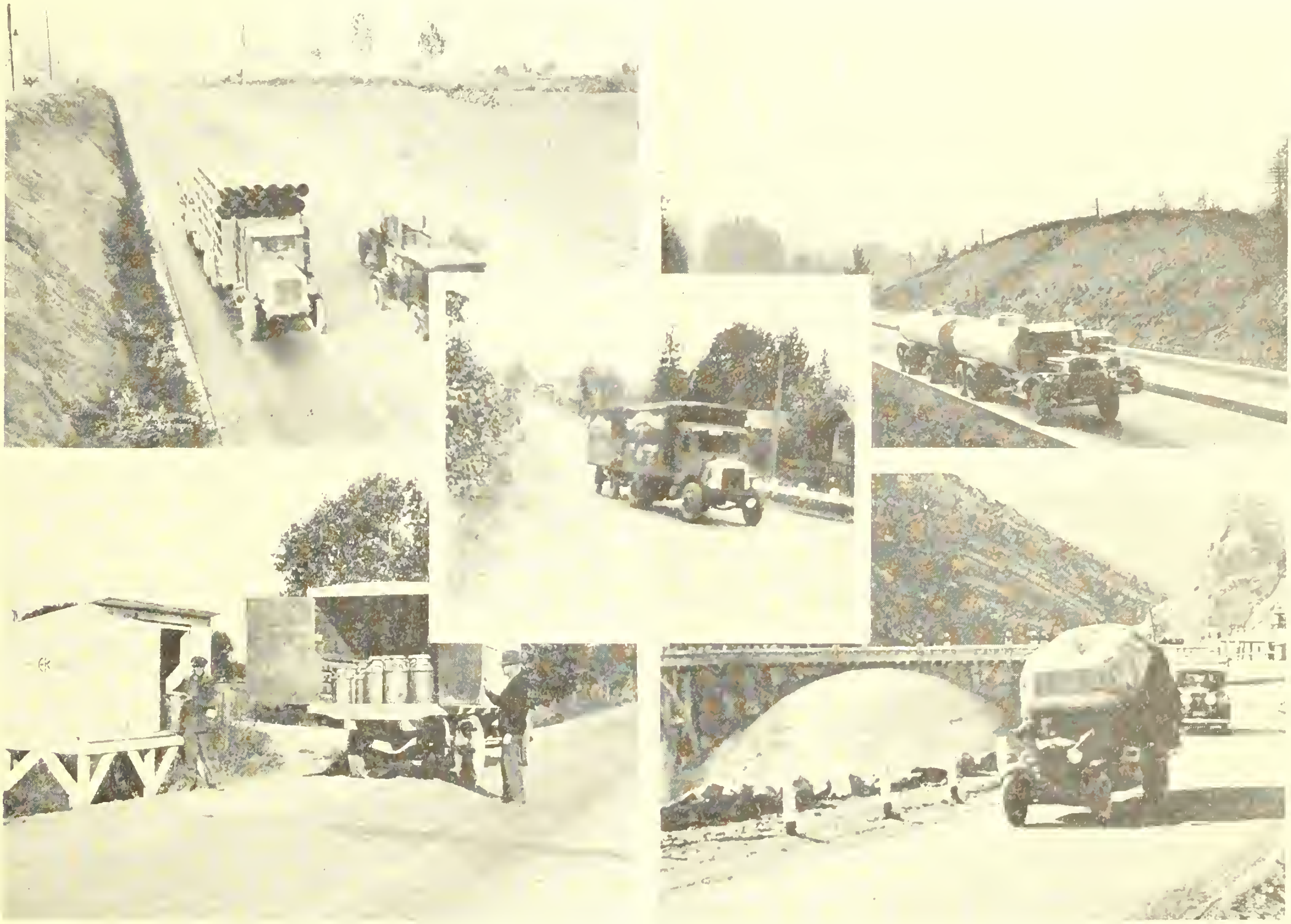


Figure 9.—Percentage Distribution of Truck Traffic by Capacity. Total Truck Traffic in Each State Represents 100 Per Cent

group in every instance is the 1 to 1½ ton group, but there are significant differences in the relative importance of light and heavy trucks in different States. The relative use of heavy trucks is greatest in California, 23.1 per cent, and least in New Mexico, the percentage being 5.2. Other States where the relative use of heavy trucks is higher than the average are Washington, Nevada, and Oregon; in Arizona, Colorado, and Idaho the percentage of heavy trucks is slightly less than the average. There is considerably less than the average percentage of heavy trucks in Nebraska, Utah, Wyoming and New Mexico.



Truck Traffic on Western Highways

Table 2.—*Percentage distribution of trucks by capacity*

Capacity group (tons)	Arizona	California	Colorado	Idaho	Nebraska	New Mexico	Nevada	Oregon	Utah	Washington	Wyoming	All States
½ to ¾	16.5	16.8	13.7	14.8	9.4	16.0	20.5	6.7	29.6	20.2	14.2	14.2
1 to 1½	48.2	41.8	59.8	57.7	65.0	63.8	45.6	56.4	47.6	44.4	64.1	53.7
2 to 2½	21.7	18.3	14.8	14.0	17.1	15.0	16.7	20.3	13.1	17.9	14.1	17.6
3 to 3½	8.9	9.7	5.1	5.4	6.0	2.9	7.7	9.1	5.7	6.9	5.3	7.3
4 to 4½	1.2	2.2	.8	1.4	1.0	.8	1.4	1.5	1.3	2.2	.5	1.4
5 to 5½	2.1	4.5	3.3	6.2	.8	.8	4.1	3.6	2.3	5.2	.9	3.2
6 to 6½	.3	1.8	1.7	.4	.2	.2	1.0	.7	0	1.7	.3	.9
7 to 7½	.2	.4	.3	.1	.1	.1	.5	.3	.2	.8	.3	.3
Over 7½	.9	4.5	.5	0	.4	.4	2.5	1.4	.2	.7	.3	1.4
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
SUMMARY OF LIGHT AND HEAVY TRUCKS												
Under 3 tons	86.4	76.9	88.3	86.5	91.5	94.8	82.8	83.4	90.3	82.5	92.4	85.5
3 tons and over	13.6	23.1	11.7	13.5	8.5	5.2	17.2	16.6	9.7	17.5	7.6	14.5
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

An indication of the daily mileages traveled by trucks is obtained by a comparison of the average and median daily mileage of trucks (Table 3). The arithmetic average for all States is 103 miles, but this average is heavily weighted by a small number of trucks which make unusually large daily mileages. Daily mileage is the distance traveled per day and is not the distance from a single origin to a single destination during that day. The median daily mileage, a distance so computed that one-half of all trucks travel less and one-half travel more than this distance, is unaffected by the smaller number of vehicles which make longer daily mileages and hence is a better measure of the average daily mileage. The median for all States is 82 miles, 21 miles shorter than the arithmetic mean. While 80 miles is not usually considered a short haul, it must be remembered that this distance is the mileage per day on rural highways, and that it usually represents one or more round trips from origin to destination.

There is an important long daily movement reflected by the 12 per cent of all trucks which travel distances in excess of 200 miles per day.

Only one-third of all foreign trucks travel less than 100 miles per day; fully one-third travel from 100 to 200 miles; while one-fifth travel between 200 and 300 miles; and one-tenth exceed 300 miles per day. As but 2.3 per cent of local trucks exceed 300 miles per day, the proportion of foreign trucks making exceptionally long trips to total foreign trucks is more than four times the proportion of local trucks making similar trips.

TABLE 3.—*Average*[1] *and median*[2] *daily mileage of trucks*

State	Foreign		Local		All trucks	
	Average mileage	Median mileage	Average mileage	Median mileage	Average mileage	Median mileage
Arizona	170	146	96	75	109	82
California	181	162	106	85	107	85
Colorado	168	150	93	74	98	77
Idaho	147	133	90	70	100	78
Nebraska	153	132	109	93	114	97
New Mexico	153	142	89	73	103	82
Nevada	136	119	76	63	87	69
Oregon	145	125	101	80	106	83
Utah	155	139	97	82	102	87
Washington	142	126	89	74	92	75
Wyoming	155	138	87	71	97	76
All survey States	154	137	98	78	103	82

[1] Arithmetic average of daily mileages of vehicles.

[2] A distance so chosen that one-half of the vehicles travel more than this distance in a day and one-half travel less.

Rural population, defined by the Bureau of the Census to include the population of villages under 2,500 as well as the population actually on farms, comprises 42.7 per cent of the total population of the 11 States. Trucks of rural ownership, as above defined, were found to include 53.3 per cent of all trucks on the highway system. Urban population, 57.3 per cent of the total, is represented by 46.7 per cent of all trucks using the highways. Thus there is definitely a greater proportionate use of the highways by rural trucks than by urban trucks. This relationship holds for all States of the survey in varying degree, except in Arizona, New Mexico, and Idaho, where the proportion of rural trucks is less than the proportion of rural population. In these three States there is a large proportion of foreign trucks which undoubtedly increase the percentages of city-owned trucks, since the majority of long-distance interstate trucking lines are city owned.

Approximately half of Arizona's foreign truck traffic originates in southern California, while a large part of the remainder is contributed by El Paso, Tex. In New Mexico one-half the foreign truck traffic originates in Texas, clearly indicating the influence of El Paso, while Colorado also contributes a large share via Walsenburg and Trinidad. The proximity of the cities of Salt Lake City, Brigham, and Ogden in Utah, and Spokane, Wash., raises the percentage of city-owned trucks traveling upon Idaho highways.

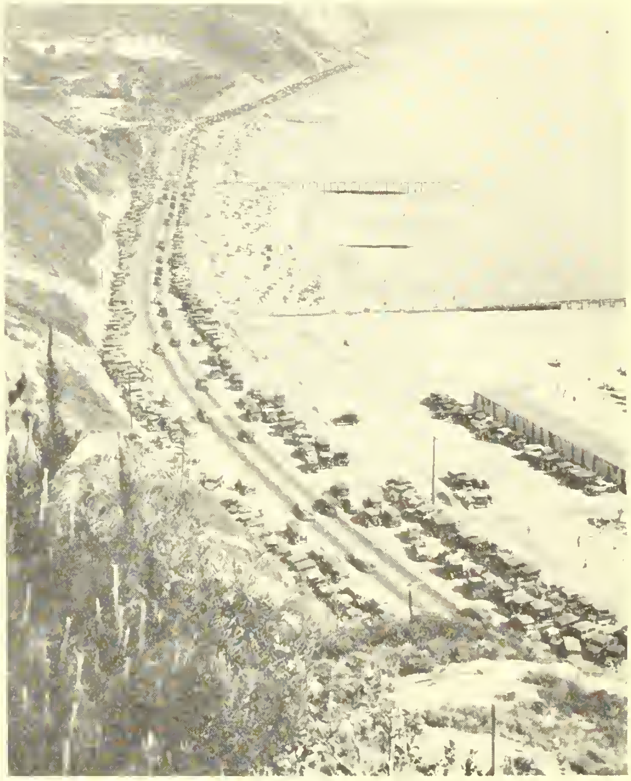
FORECASTS OF HIGHWAY TRAFFIC IN THE WESTERN STATES

Planned highway construction—widening, new construction, or reconstruction of old routes and pavements—must provide for future use as well as for present traffic.

There is little question as to the desirability of the higher types of pavement, but funds necessary for the universal construction of such types are not now available. Roads that are too highly improved are as uneconomic as those that are inadequate for traffic demands. Because of this fact, estimates of future traffic are essential in the development of a plan of highway improvement.

Traffic forecasts based solely on past traffic trends can not generally be made, as reliable traffic data for a series of years are available only in a few States. Limited past traffic data may be used in a reasonable forecast of probable future traffic where the factors of population, registration, and gasoline consumption are available. Since these needed factors are obtainable in all the Western States, a means of estimating the probable future traffic in each is present, even though no traffic series is available.

Traffic forecasts in previous reports of the Bureau of Public Roads [2] were based on estimates of registration and population. Trends of traffic and motor vehicle registration were shown for Massachusetts, Maryland, Maine, Michigan, and Wisconsin. Registration was estimated by projecting the trend of persons per car. This method was used in forecasting probable registration in the Western States.



HEAVY TRAFFIC ON CALIFORNIA 60 NORTH OF SANTA MONICA CANYON IN LOS ANGELES COUNTY

Gasoline consumption is more closely related to volume of traffic than is registration, but statistics of gasoline consumption can be obtained only for very recent years, since the institution of a fuel tax. It was therefore necessary to use a combination of these several factors.

Increased travel per car and the amount of use of a State's highways by foreign vehicles are both reflected in increased gasoline consumption within that State, although the motor-vehicle registration of the State is not affected by either of these factors. Numerous agencies of the motor-vehicle industry have published statements regarding the increasing use per car, some indicating an increased mileage per car for 1930 more than 60 per cent greater than in 1920, and these conclusions are supported by figures of increased fuel consumption per car.

The survey in the Western States (see Table 1) discloses that the range of foreign traffic is from 4.9 per cent of all traffic in California to 38.5 and 37.6 per cent in Arizona and New Mexico, respectively. The median State is Oregon, with 22.1 per cent of its traffic originating outside its boundaries. Only California has less than 10 per cent foreign traffic, while more than one-third of all traffic in Arizona and New Mexico comes from other States.

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[2] Report of a Survey of Transportation on the State Highway System of Ohio, (1927); Report of a Survey of Transportation on the State Highway System of Connecticut (1926); The Maine Highway Transportation Survey, PUBLIC ROADS, vol. 6, No. 3, May, 1925.

RELATIONSHIP OF GASOLINE CONSUMPTION, REGISTRATION, AND TRAFFIC

Since 1925 and 1926 gasoline consumption has increased more rapidly than registration in the Western States, as well as in the United States as a whole. This is clearly illustrated in Figure 10, in which the historical series of gasoline consumption, registration, and traffic have been plotted on logarithmic scale and moved together vertically to facilitate comparison of the rates of increase in two of the Western States.

Traffic data for the period 1923 to 1929 are available in Oregon. During this period registration in Oregon has increased 64.2 per cent; traffic, 101.2 per cent; and gasoline consumption by motor vehicles, 104.4 per cent. If 1929 is taken as the base year the percentage increases are as follows: Registration, 39.1 per cent; traffic, 50.3 per cent; gasoline consumption, 51.1 per cent. (Table 4.) It will be found by reference to Figure 10 that for the years from 1920 to 1924 the trends for motor vehicle registration and gasoline consumption are very similar. In 1924 these series begin to separate and the divergence continues to increase with the registration trend falling off while that of gasoline consumption continues to climb with the traffic trend.

TABLE 4.—*Comparison of motor vehicle registration, gasoline consumption, and highway traffic in Oregon, 1923–1929*

Year	Registration [1]	Index number	Gasoline consumption (gallons)	Index number	Traffic	Index number
1923	166, 412	100. 0	74, 395, 262	100. 0	86, 931	100. 0
1924	192, 629	115. 8	86, 212, 632	115. 9	110, 891	127. 6
1925	216, 553	130. 1	99, 718, 545	134. 0	120, 893	139. 1
1926	234, 134	140. 7	111, 739, 583	150. 2	137, 431	158. 1
1927	245, 705	147. 7	122, 979, 624	165. 3	146, 257	168. 2
1928	254, 415	152. 9	134, 228, 921	180. 4	163, 934	188. 6
1929	273, 270	164. 2	152, 079, 099	204. 4	174, 871	201. 2

[1] As reported to U. S. Bureau of Public Roads by State officials. In some instances a revision has been made of the figure originally reported in annual tabulations.

California has a 5-year series of traffic, gasoline consumption, and registration. Figure 10, presents trends of these factors. Here again the trends of gasoline consumption and traffic are very similar, while the registration curve, as in Oregon, is lower since 1926. During these five years, 1926 to 1930, traffic increased 39.3 per cent; registration 27.5 per cent; and gasoline consumption 40.9 per cent. If 1930 is used as a base year, the increases are as follows: Traffic, 28.2 per cent; registration, 21.6 per cent; and gasoline consumption, 29.0 per cent. Absolute and percentage figures are shown in Table 5. Further reference to California (fig. 10) brings out the interesting fact that during 1924, 1925, and 1926 gasoline consumption and registration trends were very much alike, but since 1926 they have diverged. Again the gasoline consumption trend follows more closely that of traffic.

All of the divergence of registration from traffic and gasoline consumption trends would not be accounted for by foreign traffic alone, since it amounts to but 4.9 per cent of all traffic in California. The greater use of the automobile throughout the year is another factor that is reflected in increased gasoline consumption.

TABLE 5.—*Comparison of motor vehicle registration, gasoline consumption, and highway traffic in California, 1926–1930*

Year	Registration	Index number	Gasoline consumption (gallons)	Index number	Traffic [1]	Index number
1926	1, 600, 475	100. 0	825, 106, 169	100. 0	1, 751, 799	100. 0
1927	1, 693, 195	105. 8	928, 724, 702	112. 6	2, 003, 203	114. 4
1928	1, 799, 890	112. 5	985, 558, 974	119. 5	2, 188, 741	121. 9
1929	1, 974, 341	123. 4	1, 139, 736, 244	138. 1	2, 293, 720	130. 9
1930	2, 041, 356	127. 5	1, 162, 337, 545	140. 9	2, 440, 000	139. 3

[1] This series is an average of Sunday and Monday traffic in January at 583 stations. In January, 1930, an unusually heavy snow closed many roads during the traffic count. Sunday traffic decreased more than 20 per cent, but Monday traffic, with the same roads closed, showed an increase over the previous year. A slightly lower rate of increase than that of July, 1930, over July, 1929, traffic was used to interpolate the January, 1930, figure. Had the full rate of increase in July traffic been applied, the index number for traffic in 1930 would have been closer to the gasoline consumption index of 140.9, i. e., 140.3 instead of 139.3.

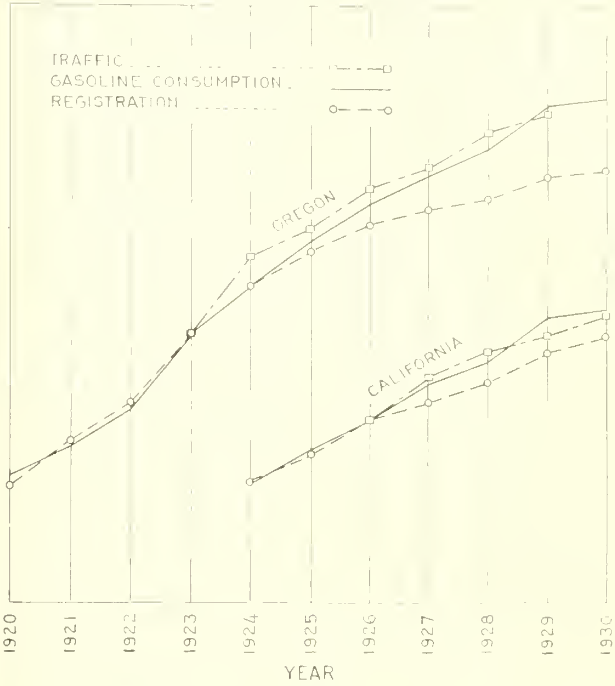


FIGURE 10.—TRENDS OF HIGHWAY TRAFFIC, GASOLINE CONSUMPTION, AND MOTOR VEHICLE REGISTRATION IN OREGON AND CALIFORNIA. THE VERTICAL SCALE IS LOGARITHMIC AND THE CURVES HAVE BEEN MOVED TOGETHER VERTICALLY FOR COMPARISON OF RATES OF INCREASE

Table 6, based on data for the State of Louisiana, further illustrates the fact that traffic is increasing at a faster rate than registration. It should be noted that gasoline figures in Louisiana do not represent gallons of gasoline consumed by motor vehicles as in Oregon and California, but total gallons for all purposes. Louisiana traffic increased 25.2 per cent from 1926 to 1930. Registration increased 14.9 per cent and gasoline consumption increased 36.4 per cent. Fragmentary data from Nebraska, where a small number of traffic stations were operated at changing locations from 1926 to 1930, indicate closer agreement in rates of increase of

TABLE 6.—*Comparison of motor vehicle registration, gasoline consumption, and highway traffic in Louisiana, 1926–1930*

Year	Registration	Index number	Gasoline consumption (gallons)	Index number	Traffic	Index number
1926	239, 500	100. 0	135, 428, 367	100. 0	104, 710	100. 0
1927	255, 000	106. 5	151, 702, 807	112. 0	116, 270	111. 0
1928	264, 293	110. 4	169, 046, 556	124. 8	116, 575	111. 3
1929	280, 868	117. 3	176, 645, 631	130. 4	130, 270	121. 4
1930	275, 283	114. 9	184, 781, 753	136. 4	131, 097	125. 2

traffic and gasoline consumption than in rates of increase of traffic and motor vehicle registration.

Statistics of gasoline consumption in the State of Colorado are available for a longer period of years than in any other State, i. e., since 1913. The rates of increase in motor vehicle registration and in gasoline consumption are very similar year by year between 1913 and 1925. After 1925 the increases in registration take place at a less rapid rate than the rates of increase in gasoline consumption for corresponding years. The figures of Table 7 present the data for comparison.

Table 7.—*Comparison of registration of motor vehicles and gasoline consumption in Colorado, 1913–1930*

Year	Registration [1]	Gasoline consumption: Net [1] (gallons)	Gasoline consumption: Gross [1] (gallons)	Index numbers: Registration	Index numbers: Gasoline (net)	Index numbers: Consumption (gross)
1913	13,000	------	5,860,855	5.4	------	5.9
1914	17,756	------	10,372,238	7.4	------	10.5
1915	28,894	------	14,482,629	12.0	------	14.7
1916	43,296	------	19,988,001	18.0	------	20.2
1917	87,460	------	29,879,153	36.4	------	30.3
1918	83,244	------	32,800,910	34.7	------	33.2
1919	104,865	------	42,361,550	43.7	------	42.9
1920	129,255	[2] 45,839,482	51,917,098	53.8	47.1	52.6
1921	115,739	[2] 64,491,230	60,390,692	60.7	66.2	61.2
1922	162,328	69,337,164	65,891,200	67.6	71.2	66.7
1923	188,956	73,753,225	75,258,403	78.7	75.7	76.2
1924	213,247	92,151,131	94,031,766	88.8	94.6	95.2
1925	240,097	97,377,858	98,741,301	100.0	100.0	100.0
1926	248,613	104,587,460	112,380,309	103.6	107.4	113.8
1927	268,492	122,493,107	128,304,024	111.8	125.8	129.9
1928	284,867	130,707,467	142,027,665	118.7	134.2	143.8
1929	303,489	141,466,891	155,507,842	126.4	145.3	157.5
1930	308,509	153,620,645	170,854,939	128.5	157.8	173.0

[1] As reported by State Inspector of Oils.

[2] Based on taxes collected in 1920 apparently reported in part with collections of 1921.

EFFECT OF PRESENT DEPRESSION ON FORECASTS IN WESTERN STATES

General trend lines were considered in arriving at figures in the Western States forecasts. Previous business slumps have checked the rate of decline in persons per car for a year or two, and in a few States for a longer period, but this ratio has always returned to its general trend, as is clearly illustrated in Figure 11.

Statistics show that while registration barely held its own in 1930, gasoline consumption and traffic have had definite increases.

MILES PER GALLON

Where data have been obtained on the operation of any considerable number of passenger cars it has been found that the average mileage per gallon is around 15 miles. Some individual records are as high as 23 miles per gallon, others as low as 11.

Some investigators have suggested 11,000 miles as an average annual mileage. Others indicate that the figure is much lower, from 6,000 to 10,000 miles.

Assuming 8,000 miles per year as the average travel of registered vehicles in the United States and dividing the number of gallons of gasoline consumed into the vehicle mileage, we obtain an average of 14.38 miles per gallon as the average rate of fuel consumption. Were the 11,000 miles per vehicle used, the result would be 19.78 miles per gallon for all vehicles.

The rapid increase in trucks registered during the decade 1920 to 1930 (246.0 per cent net) as compared to passenger cars registered (180.1 per cent net) very probably has had considerable effect upon miles per gallon for all vehicles. However, trucks amounted to 10.9 per cent of total registration in 1920, and 13.1 per cent in 1930. If 15 miles per gallon were used for passenger cars and 8 miles per gallon for trucks, the weighted average in 1920 would be 14.24 miles per gallon as compared with 14.08 in 1930.

In a truck survey by the General Motors Corporation involving 46,000 trucks, the miles per gallon for light, medium, and heavy trucks are given. When weighted by per cent of each capacity, the average for all is 11.25 miles per gallon. Using 11.25 for trucks and the above 15 for passenger cars, the weighted average in 1920 was 14.59, and in 1930, 14.51 miles per gallon. In one case, the decrease was sixteen-hundredths, and the other eight-hundredths of a mile per gallon in the past 10 years. The point is that truck registration (including busses) is so small as compared to total registration that the reduction of miles per gallon for all vehicles is small indeed.

This small change in average mileage per gallon is an indication that little additional traffic may be estimated from this factor.

PERSONS PER MOTOR VEHICLE

The number of persons per car was found for each State from 1913 to 1930 and extended to 1940, based on population as of July 1. These data are shown graphically in Figure 11 for the years since 1914, 1915, or 1916. The number in 1913 is often more than double the 1915 or 1916 figure, and can not conveniently be shown on these charts.

The curves shown in Figure 11, are very similar in that during the first five or six years the decline was rapid. Also in most States, during 1921 and 1922, the descent of these trends was checked sharply, after which the curves became smooth and tended to flatten out more and more each year. Because of business conditions, the number of persons per car in 1930 was affected somewhat as in 1921 and 1922. States with large deviations from trend are as follows: Wyoming and New Mexico for 3 years; Idaho for 5 years; Arizona and Nevada for 2 years. Practically all States had a high or low registration during one of the years 1917, 1918, or 1919, a fact which caused persons per car to be above or below the general trend in one of these years.

Registration, therefore, during years of depression may be low when compared with the general trend. It is interesting to note the rapid recovery in most States in 1923 after the very sharp deviation from trend in 1921 and 1922.

The traffic forecasts were based largely upon projected increases in gasoline consumption. Consideration was given to the nature of the data on gasoline consumption (whether net or gross), to the amount of foreign traffic recorded within the State, and to local distribution of registration, gasoline consumption, and foreign traffic, before a specific forecast was made for a particular State. Each State was subdivided into two or more geographical sections according to density of population; and a forecast was made of the increase in traffic on the highway system within each area, on the basis of the projected increase in gasoline consumption, as modified by the considerations outlined above. Motor vehicle registration by counties was available in 9 of the 11 States, gasoline consumption by counties in but 4 States.

The estimated increases in traffic from 1930 to 1940 vary between 42 per cent in eastern Washington to 76

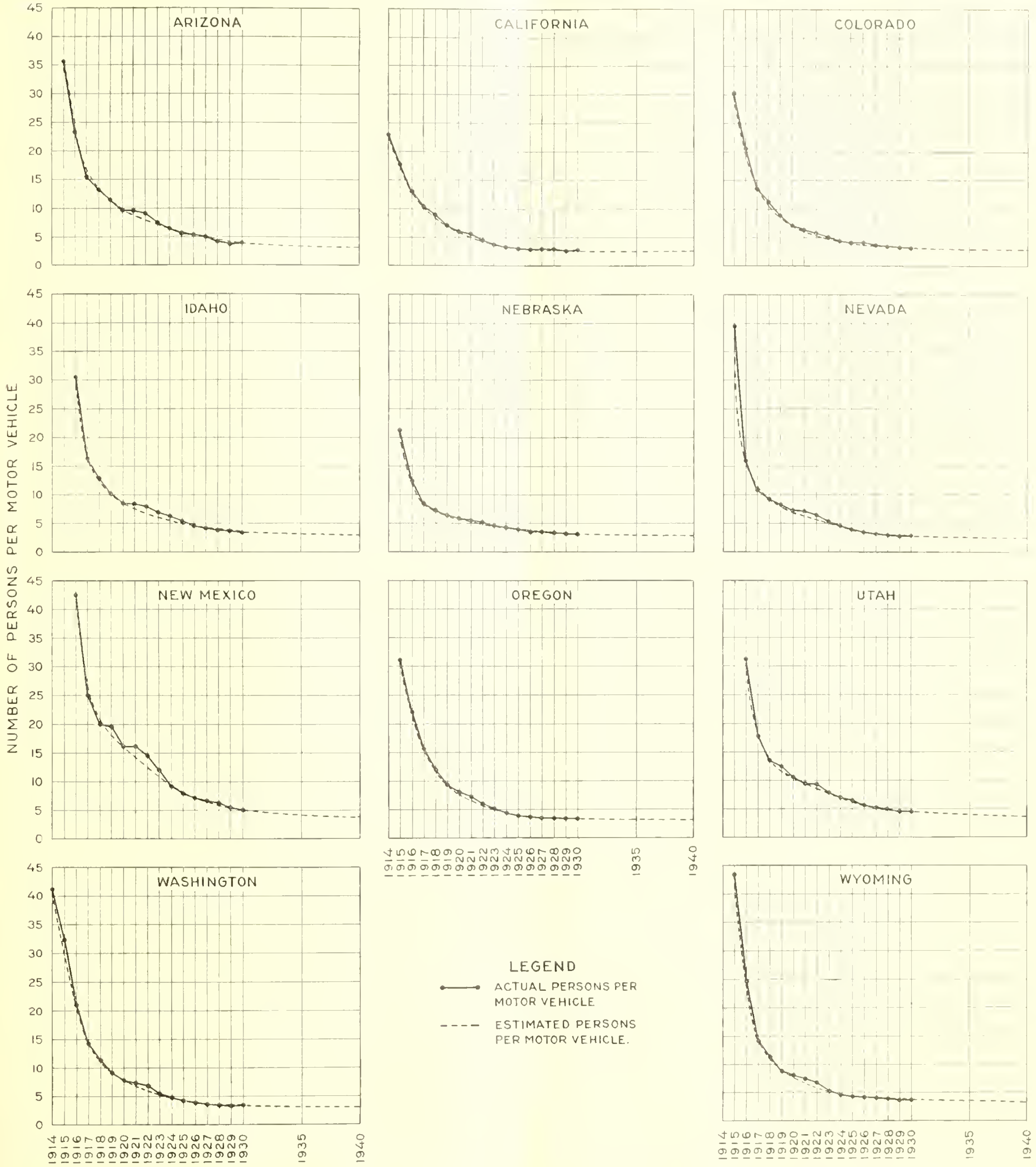


FIGURE 11.—CURVES SHOWING NUMBER OF PERSONS PER MOTOR VEHICLE IN WESTERN STATES DURING YEARS BETWEEN 1914 AND 1930. CURVES ARE PROJECTED TO SHOW PROBABLE VALUES FOR 1935 AND 1940

per cent in northern Arizona. For most of the area the traffic forecast varies between 45 and 60 per cent.

TRAFFIC CLASSIFICATION OF FEDERAL-AID SYSTEM IN WESTERN STATES

In the determination of a consistent program of economical highway improvement it is essential to consider the present traffic and also the traffic anticipated throughout the life of the proposed improvement, or as far ahead as reasonably accurate forecasts may be made. For this purpose the highway systems of the 11 survey States are classified in three groups according to average daily traffic density. A traffic of 1,500 or more vehicles per day is classified as heavy, 600 to 1,500 as intermediate, and under 600 as light. For the purposes of the forecast the classification is subdivided into groups A to G, as shown in Table 8.

TABLE 8.—*Scheme of classification for highways of Western States according to average daily traffic density in 1930 and predicted density in 1935 and 1940*

Classification	1930	1935	1940
A	Over 1,500	Over 1,500	Over 1,500
B	600–1,500	Over 1,500	Over 1,500
C	600–1,500	600–1,500	Over 1,500
D	600–1,500	600–1,500	600–1,500
E	Under 600	600–1,500	600–1,500
F	Under 600	Under 600	600–1,500
G	Under 600	Under 600	Under 600

Sections of the system carrying an average traffic of 1,500 or more vehicles per day, or more than 30 heavy trucks per day when the total volume of traffic is close to 1,500, are classified as heavy in 1930. Those expected to carry this volume by 1935 or 1940 are classified as heavy in 1935 or 1940, respectively. Similarly, those sections carrying an average of between 600 and 1,500 vehicles per day are classified as intermediate in 1930, with corresponding classifications for additional sections in 1935 and 1940, as the predicted increase in traffic brings them within this range. Those sections carrying an average of less than 600 vehicles per day in 1930, 1935, or 1940 are classified as light traffic highways in the respective periods.

The classification covers the Federal-aid system as of June 30, 1930, and is based on mileage figures submitted by the various States. The routes are carried continuously through all cities regardless of population.

The distribution of road mileage in the Western States according to the classification adopted is shown in Figure 12. On this map roads or sections of roads falling into each of the eight classifications (A to G) given in Table 8 are designated by appropriate symbols. Those roads which it was found impossible to classify are shown in double dash line.

The mileage included in each classification group by individual States is summarized in Table 9, which indicates the general increase of heavy and intermediate traffic routes, the heavy traffic mileage increasing from 11.3 per cent of the total mileage in 1930, to 14 per cent in 1935, and to 16 per cent in 1940. Similarly the total of the heavy and intermediate routes increases from 29.3 per cent of the total mileage in 1930, to 36.1 per cent in 1935, and to 41 per cent in 1940.

Practically half the class A highways (heavy traffic in 1930), are found in California, with Washington and Oregon adding some 1,200 miles. The three coast States contain more than three-quarters of all the class A mileage. The remainder is distributed throughout the other States, in large degree according to the size and location of the centers of population.

The mileage coming into the heavy-traffic classification in 1935 and 1940 is by no means proportional to the mileage now included in this classification in the various States. According to the forecast Nebraska, Nevada, and New Mexico will practically quadruple their present heavy traffic mileage by 1940, while Wyoming, which now has none, will have 68 miles by that time. California and Washington, with their present large mileage of heavy traffic roads, show relatively little increase. Although Utah is low in class A mileage, present traffic is so concentrated that practically no increase in its heavy traffic mileage is likely to occur by 1940. Oregon, with 339 miles in classes B or C in 1930, adds the greatest mileage to its heavy traffic classification between 1930 and 1940.

TABLE 9.—*Mileage* [1] *of Federal-aid roads in the Western States falling within heavy, intermediate, and light classifications, on the basis of average daily traffic density in 1930 and predicted density in 1935 and 1940*

HEAVY

State	1930		1935		1940	
	Miles	*Per cent*	*Miles*	*Per cent*	*Miles*	*Per cent*
Arizona	115.1	5.8	165.9	8.3	178.9	9.0
California	2,015.9	39.5	2,272.1	44.5	2,339.1	45.8
Colorado	334.9	10.3	382.4	11.8	528.0	16.3
Idaho	109.6	3.5	155.1	5.0	221.2	7.1
Nebraska	66.4	1.1	190.9	3.3	245.7	4.2
Nevada	12.1	.8	29.5	1.9	50.1	3.2
New Mexico	25.5	.7	69.5	2.0	118.4	3.4
Oregon	440.5	13.3	581.8	17.5	779.6	23.4
Utah	169.9	9.6	172.7	9.8	172.7	9.8
Washington	772.4	24.6	949.8	30.3	1,030.9	32.9
Wyoming			52.8	1.5	67.9	1.9
Total	4,062.3	11.3	5,022.5	14.0	5,732.5	16.0

INTERMEDIATE

State	1930		1935		1940	
	Miles	*Per cent*	*Miles*	*Per cent*	*Miles*	*Per cent*
Arizona	399.4	20.0	670.6	33.7	903.9	45.4
California	1,130.8	22.1	1,042.8	20.4	1,115.9	21.8
Colorado	722.5	22.4	925.8	28.6	1,091.1	33.7
Idaho	355.7	11.5	475.5	15.4	520.8	16.9
Nebraska	1,482.1	25.5	1,830.6	31.5	1,974.2	34.0
Nevada	55.2	3.5	90.2	5.7	80.6	5.1
New Mexico	321.2	9.3	660.2	19.2	953.4	27.8
Oregon	796.2	23.9	926.4	27.9	886.6	26.8
Utah	75.1	4.3	149.3	8.4	201.5	11.3
Washington	898.3	28.7	795.0	25.3	866.6	27.6
Wyoming	228.1	6.5	378.7	10.9	404.1	11.7
Total	6,464.6	18.0	7,945.1	22.1	8,998.7	25.0

LIGHT

State	1930		1935		1940	
	Miles	*Per cent*	*Miles*	*Per cent*	*Miles*	*Per cent*
Arizona	1,441.4	72.5	1,119.4	56.3	873.1	43.9
California	1,857.5	36.3	1,689.3	33.0	1,549.2	30.3
Colorado	2,130.8	65.7	1,880.0	58.0	1,569.1	48.4
Idaho	2,343.6	75.9	2,178.3	70.5	2,066.9	66.9
Nebraska	3,449.7	59.5	2,976.7	51.3	2,778.3	47.9
Nevada	1,370.3	87.5	1,317.9	84.2	1,306.9	83.5
New Mexico	3,090.6	90.0	2,707.6	78.8	2,365.5	68.8
Oregon	1,995.0	60.2	1,723.5	52.0	1,565.5	47.2
Utah	1,518.3	85.4	1,441.3	81.1	1,389.1	78.2
Washington	1,368.3	43.6	1,294.2	41.3	1,141.5	36.4
Wyoming	3,109.9	89.9	2,906.5	84.0	2,866.0	82.8
Total	23,675.4	65.9	21,234.7	59.1	19,471.1	54.2

UNCLASSIFIED

State	1930		State	1930	
	Miles	*Per cent*		*Miles*	*Per cent*
Arizona	33.7	1.7	Oregon	86.8	2.6
California	105.9	2.1	Utah	12.0	.7
Colorado	50.8	1.6	Washington	97.6	3.1
Idaho	281.9	9.1	Wyoming	125.3	3.6
Nebraska	803.4	13.9			
Nevada	128.4	8.2	Total	1,725.8	4.8
New Mexico					

[1] Mileage as of June 30, 1930.

The distribution of the intermediate-traffic mileage is entirely different from that of the heavy-traffic mileage. Nebraska, with its more evenly distributed population and greater mileage of highway per square mile, leads in the intermediate-traffic mileage, with nearly 1,500 miles now in this classification. This class in Nebraska also shows a marked increase in 1935 and 1940, the mileage changing from the light to the intermediate classification far exceeding that changing from the intermediate to the heavy classification. This fact is true in a greater or less degree in every State except California and Washington.

The light-traffic highways decrease in all of the States during the 10-year period. California shows the

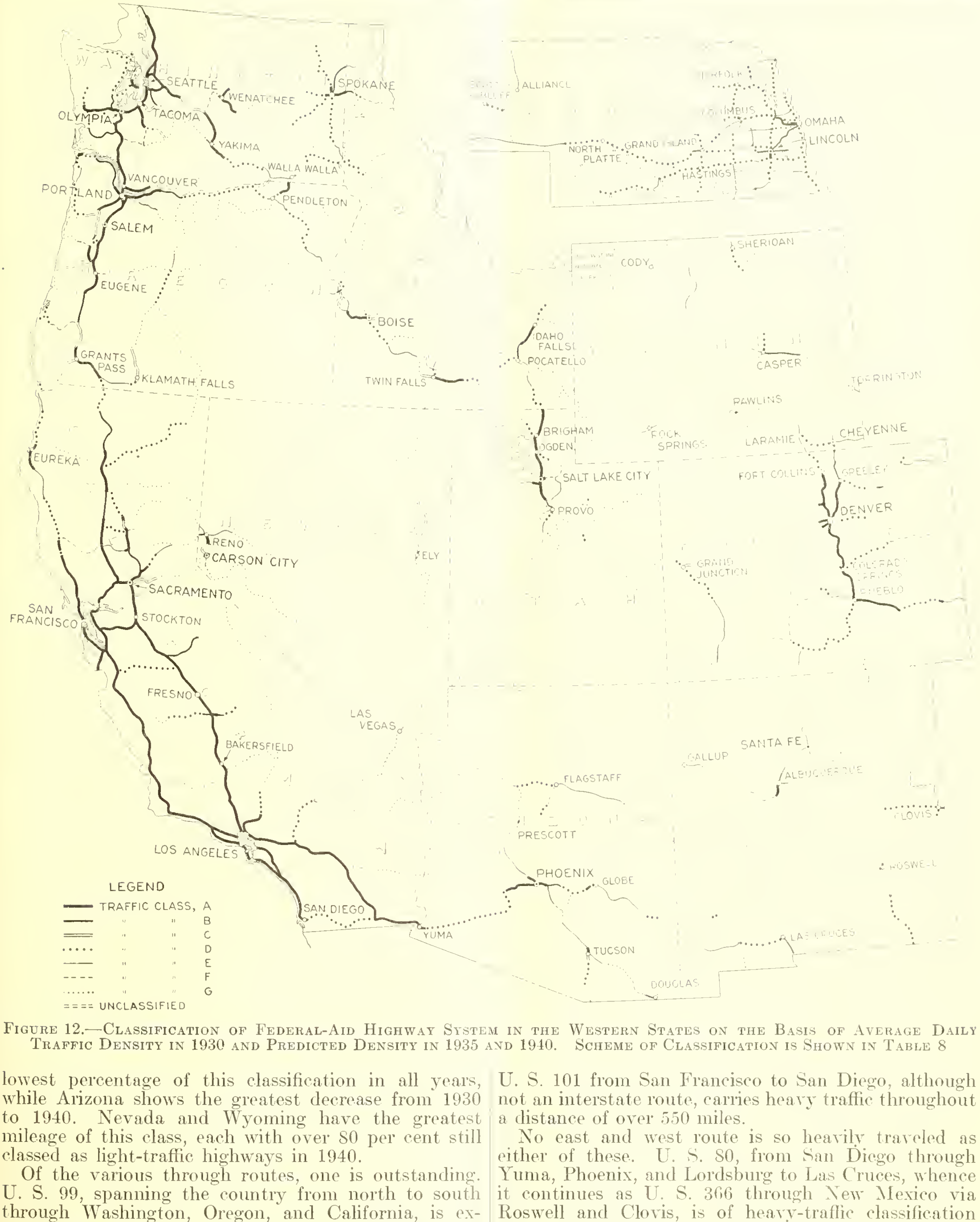


Figure 12.—Classification of Federal-Aid Highway System in the Western States on the Basis of Average Daily Traffic Density in 1930 and Predicted Density in 1935 and 1940. Scheme of Classification Is Shown in Table 8

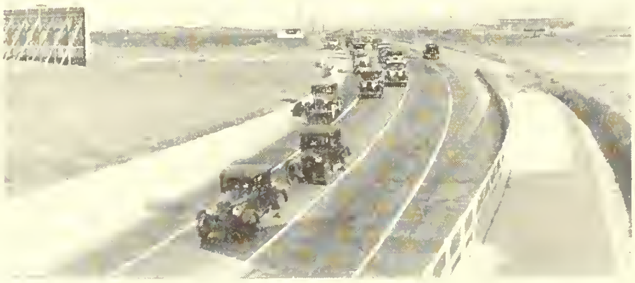
lowest percentage of this classification in all years, while Arizona shows the greatest decrease from 1930 to 1940. Nevada and Wyoming have the greatest mileage of this class, each with over 80 per cent still classed as light-traffic highways in 1940.

Of the various through routes, one is outstanding. U. S. 99, spanning the country from north to south through Washington, Oregon, and California, is expected by 1935 to carry heavy traffic throughout its entire length, with the exception of a short section at the Oregon-California line. This route now carries heavy traffic from Bellingham, Wash., to Drain, Oreg., and through California from Willows to El Centro. U. S. 101 from San Francisco to San Diego, although not an interstate route, carries heavy traffic throughout a distance of over 550 miles.

No east and west route is so heavily traveled as either of these. U. S. 80, from San Diego through Yuma, Phoenix, and Lordsburg to Las Cruces, whence it continues as U. S. 366 through New Mexico via Roswell and Clovis, is of heavy-traffic classification in 1930 only in southern California and in the vicinity of Phoenix. The traffic by 1940 will probably be such as to place practically the entire route in the intermediate or higher classification. Another relatively heavily traveled east-west route is U. S. 30, from

Omaha through Nebraska, Wyoming, Idaho, and Oregon to Astoria. The section from Omaha to Laramie is now, or by 1935 will be, classed as heavy or intermediate. From Laramie through Wyoming the route will not leave the light classification by 1940, with the exception of short sections near the various cities; but from McCammon, Idaho, through that State and Oregon to Astoria, all but a small portion will be in either the intermediate or heavy classification by 1935.

U. S. 40, the northern route to California, through Colorado, Utah, and Nevada, carries but little traffic based on the year-round average. The only sections of this route carrying traffic above the light classification are those in Colorado from the Kansas line to Denver, in the vicinity of the cities of Salt Lake City and Reno, and in California from Emigrant Gap to San Francisco.



Sunday Afternoon Traffic on Bayshore Highway Near South San Francisco

The only other through route carrying noticeably heavy traffic is U. S. 85 from El Paso through New Mexico, Colorado, and Wyoming. Over a good portion of this route—from Los Lunas, N. Mex., through Colorado to Cheyenne, Wyo.—the traffic falls in the intermediate or heavy classification.

Other routes or sections of routes carrying heavy or intermediate traffic can nearly all be found either within or connecting various economic areas. An examination of the traffic classification map (fig. 12) shows these areas well defined around Salt Lake City, Seattle, Spokane, San Francisco, and Los Angeles, and including eastern Nebraska, eastern Colorado, southern Idaho, and western Oregon and Washington. The location and extent of these routes show the insignificance of State lines or other artificial boundaries in determining traffic flow. The routes may be seen extending from State to State with little or no change in traffic classification, but changing noticeably as they progress from a center of population, or approach a natural barrier to travel.

A study of the changes in traffic classification during this 10-year period indicates its usefulness in the formulation of a long time plan of highway improvement.

PREPARATION OF PROGRAMS OF ROAD CONSTRUCTION

The traffic data provided by the survey may be used in each State as the basis for the preparation of a program of road construction, reconstruction, and maintenance consistent with traffic requirements during the ensuing 10-year period. For this purpose the characterization of traffic as heavy, intermediate, and light, and the classification of the various sections of highway according to the character of their present and probable future traffic density, are especially useful.

Traffic density is the most important general factor in highway planning. It is the multiplier that determines the amount of vehicular operating savings resulting from road improvements; and, particularly with respect to low and intermediate types of road surface, it has a determining effect upon the life and maintenance cost of the surface. It also influences strongly the choice of surface width and others of the many decisions that must be made in the development of the highway plan.

It is a recognized principle of highway finance that the expenditure for road improvement should be kept within the earning capacity of the improvement. The return to the public upon its highway investment takes the form of reductions in the operating cost of vehicles resulting from the greater ease of traction over the improved grades and surfaces and the reduced wear and tear of vehicles consequent upon the road improvement. With increase in the movement of vehicles there is greater accumulation of individual savings and increase of the amount that may justifiably be expended to obtain further benefits.

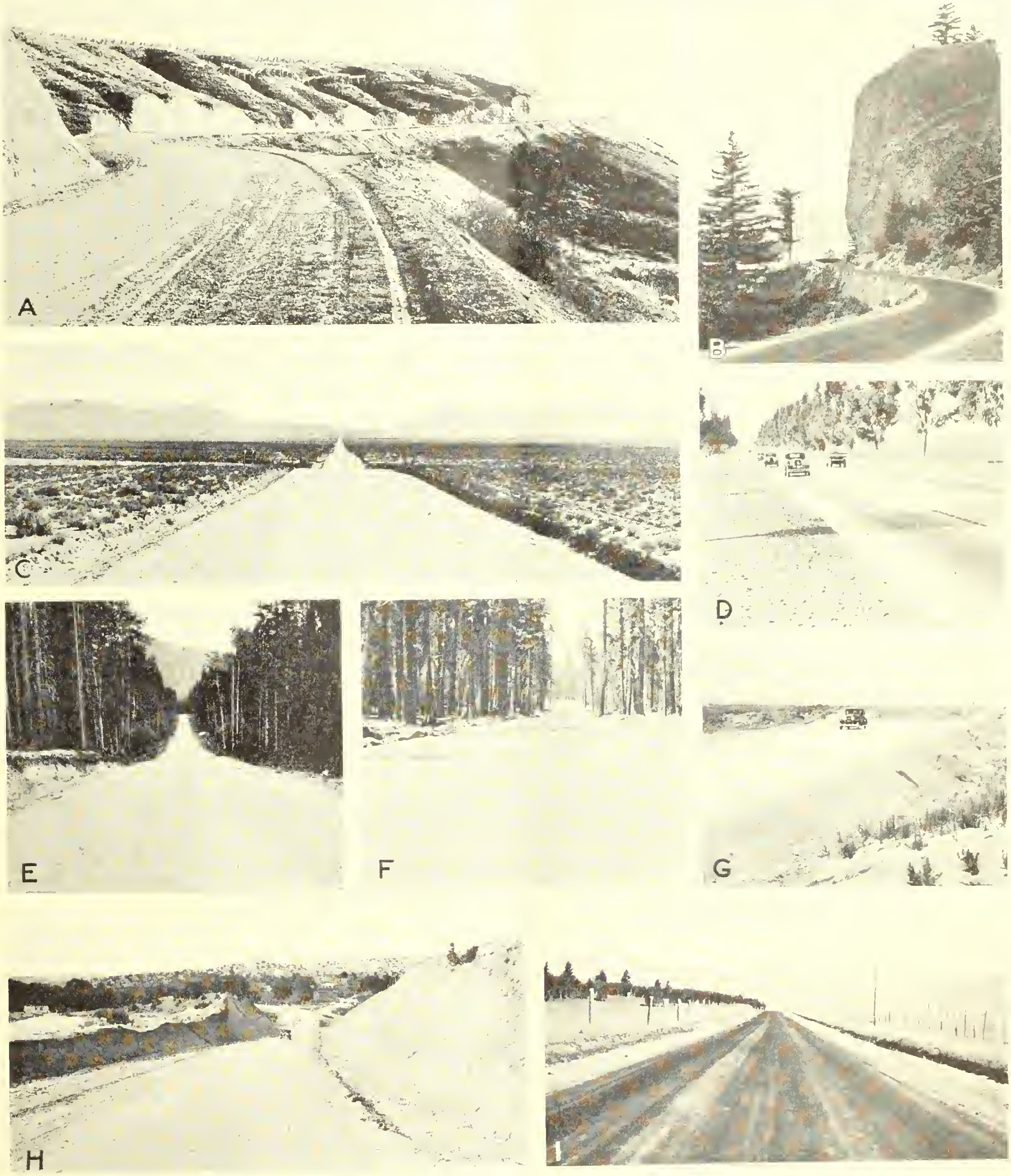
The traffic data supplied by this survey and the estimates of future traffic density based upon them afford the needed safeguard against unwise overexpenditure by indicating the amount of the vehicular operating saving that may be expected from the improvement of each section of road during the next 10 years. But the more positive value to the highway designer lies in the usefulness of the data, when intelligently employed, in determining the character of improvement required immediately by each section of the highway system and the future alterations in the form of present improvements that will probably be required by changes in the density of traffic during the 10-year period covered by the estimates.

The area covered by the survey is economically youthful. Traffic upon many of its roads is in an early stage of development and, at its present density, requires and will yield a compensatory return upon only a minimum improvement of the road surface. As traffic increases a point will be reached at which, because of the wear and cost of maintaining the surface, it will become cheaper to replace the initial low-type surface with a surface of intermediate cost and resistance to traffic. At the same time the greater savings accumulated by the increased operation of vehicles will support the greater investment in the higher type of road improvement. There are numerous sections on which the traffic, as indicated by the survey, has already reached the density that suggests such an intermediate type of road improvement.

With further increase of traffic the roads now or subsequently to be improved with intermediate types of surface will require for maximum economy a still further improvement by addition of high-type pavements; and again, as the data of the survey show, there are already many sections that have reached this degree of utilization.

It is not possible to fix upon any precise density of traffic, uniformly acceptable under the varied conditions obtaining in the several States, as the density at which a substitution of an intermediate for a low type of surface or a high for an intermediate type will become profitable.

The proper time for change is indicated by increase of the true annual cost of the lower type of surface with increase of traffic to an amount exceeding the estimated



Typical Western Road Surfaces

A—Gravel in Wyoming. B—Bituminous Concrete in Oregon. C—Gravel in Nevada. D—Portland Cement Concrete in California. E—Crushed Rock in Idaho. F—Bituminous Surface Treatment in California. G—Crushed Caliche in New Mexico. H—Graded Earth in New Mexico. I—Oil-Treated Gravel in Idaho

annual cost under the same density of traffic for the higher, more resistant, and more expensive type of surface. But the annual costs of the various types of surface under various densities of traffic are affected both absolutely and relatively by the different conditions of the various parts of an area of such diverse conditions as the wide territory covered by this survey. Even within an individual State it may be impracticable to base highway design upon a single relation of traffic density and road type. Where, as in this case, there are 11 States and a range of natural conditions from mountain to plain, from arid to humid, from cold to hot, it is quite impossible to apply a uniform rule.

To illustrate, consider the traffic suitability of the gravel surface, a familiar and widely used low type. The annual cost of a gravel surface involves first the sum derived by dividing the difference between the cost of construction and the salvage value at the time of replacement by the time in years between construction and reconstruction. To this is added the annual interest on the capital invested and the annual cost of maintenance under the traffic to which the surface is actually subjected.

Obviously this annual cost will be affected by differences in the cost of the gravel and by differences in the character of the gravel which will cause the surface to wear more or less rapidly under traffic of various intensities and thus influence the life of the surface. The character of the subgrade upon which the surface is laid and the general climatic conditions obtaining will also have important bearing, as will several other variable conditions.

Similar variable conditions also affect the annual cost of the higher types of surface that may be considered for substitution in place of the gravel surface. So that it is impossible to fix upon any single density of traffic that may be uniformly used throughout such an area as the territory of this survey as the criterion of change from gravel to bituminous macadam or other higher type of surfacing.

For reasons such as this it is impossible to attempt in this report to establish the desirable highway program in all the 11 States. The current traffic data and the estimates of future density supplied constitute the principal basis of such programs; but the establishment of the programs is a task that must be left to the several State highway departments familiar with the present condition of the various highways and the various conditions, such as the cost and character of available road materials, and the effect of the existing climate and soils upon surfaces built of such materials.

Moreover, there will be special cases in which traffic density or tangible economy will not be controlling factors in determining the type of road improvement. Such cases may be expected to be rather more numerous in these Western States than in other more fully developed areas. In remote mountain and desert regions, for example, it is quite possible that the type of road improvement may be determined by such factors as the convenience, safety, comfort, and speed of traffic to a greater degree than by the more tangible factors of traffic density and cost.

In all cases, however, the tables and maps contained in the report, showing as they do the average daily density of traffic upon all sections of the Federal-aid highway system at the time of the survey and the estimated density after 5 and 10 years, respectively, will serve as a reliable guide in the establishment of the highway program. The characterization of the traffic as light, intermediate, and heavy according as its average daily density is less than 600, between 600 and 1,500, and over 1,500 vehicles, respectively, and the classification of the various road sections according to this denomination of their traffic at the three periods, represent efforts to reduce the data to a practically usable form. The limits assigned to the three classes of traffic, that is, 600 and 1,500 vehicles per day, are not to be construed as traffic densities indicative of the need of low, intermediate, or high type surfaces; although, in the general way possible under the circumstances, modified as necessary by other known facts, the passage of a section of road from one to the other of the resulting classes may be accepted as an index of traffic growth critical in its bearing upon highway design.

TRUCK GREAT FACTOR IN FARM FREIGHTING

MOTOR trucks are hauling approximately 15 per cent of the total shipments of fresh fruits and vegetables transported 20 or more miles to market, according to a survey made recently by the Bureau of Agricultural Economics. Motor-truck shipments of 20 or more miles in 1929 are estimated by the bureau at between 150,000 and 200,000 cars, as compared with 1,068,745 cars transported by rail and boat lines. This represents a large percentage increase in motor-truck hauls in recent years, but it is pointed out that rail and boat shipments have practically doubled in the last 10 years. On a mileage basis the percentage of motor trucks to total shipments is much less than 15 per cent because of the longer average haul by railroads.

In areas covered by the survey motor-truck hauls ranged from 2 to 92 per cent of the shipments which went 20 or more miles in those areas in 1928, the high figure being for the State of Connecticut. In Southwestern Michigan 73 per cent of the shipments going 20 or more miles to market go by motor truck; on Long Island, 68 per cent; in the Hudson Valley, 67 per cent; central and southern New Jersey, 67 per cent; Delaware, 41 per cent; southern Indiana, 37 per cent; eastern shore of Maryland, 24 per cent; western Maryland, 24 per cent; southern Illinois, 21 per cent; western New York, 19 per cent; south central Pennsylvania, 18 per cent; western Massachusetts, 7 per cent; eastern shore of Virginia, 3 per cent; and the West Virginia, Cumberland, Shenandoah Valley region, 2 per cent. Motor-truck hauls in these areas totaled 77,102 cars,as against 136,509 cars carried by rail and boat. In important producing areas remote from large markets—Florida, southern Texas, California, the Northwestern States, and northern Maine—the percentage of shipments hauled by motor truck is much smaller than in some of the areas covered by the survey.

The bureau investigated truck receipts at city markets and trade and operating practices of truckmen, farmers, truckmen carriers, and truckmen merchants, and studied the economic aspects of shipping by truck in their effect upon distribution and production.

The following is a summary of some of the information brought out by the survey:

Trucks have expedited transportation on short hauls and have made the distribution of highly perishable products more direct and less wasteful under certain conditions.

Regional motor-truck jobbing markets, where products from a considerable area are concentrated and redistributed, and wholesale roadside stands, are increasing in number to serve the motor-truck trade.

Products most suited to long-distance transportation by motor truck are the light, quickly perishable fruits and vegetables or those that yield a high freight revenue and require expeditious movement to market.

Distances covered by trucks have increased; highly perishable products are now being hauled regularly for distances as great as 400 miles, and even greater distances in some areas.

For all the areas in north central and northeastern United States that were studied the relation which motor-truck shipments of important commodities bore to the total shipments was, in terms of percentage: Spinach, 96; snap beans, 89; mushrooms, 85; asparagus, 76; tomatoes, 64; strawberries, 58; cantaloupes, 49; grapes, 48; peaches, 43; potatoes, 25; apples, 24; lettuce, 21; sweet potatoes, 19; onions, 18; and cabbage, 12.

The truck unloads of fruits and vegetables at the 11 cities studied, excluding market-garden receipts, were of greatest relative importance at Los Angeles where they were 57 per cent of the total of unloads for the city, and of least relative importance at Boston, where they were only 2 per cent of the total unloads.

Redistribution from city markets to surrounding trade territories has grown in volume and in distance. Except for local supplies, the area within 50 miles is now usually supplied with fruits and vegetables by truck from the large city market. In the outer rim of the trade territory, up to 150 miles, and sometimes farther, trucks compete with mixed cars and express shipments from the large city and with straight cars shipped direct from producing areas.

It was estimated that 50 per cent of the total receipts on the wholesale markets at Pittsburgh was trucked out of the metropolitan area in the spring of 1930, and at Baltimore 24 per cent was trucked out.

Country cold-storage plants aid motor-truck transportation by prolonging the trucking season. The use of the truck facilitates quick movement of apples into storage after packing, which is a decided advantage. Canning-plant managers are using trucks to obtain more soft fruits and tomatoes of desirable maturity and to extend the area from which supplies are received.

PROGRAMS OF ESTIMATED STATE AND LOCAL HIGHWAY AND BRIDGE EXPENDITURES FOR CALENDAR YEAR 1932

(*As reported by State authorities*)

State	Grand total expenditures (estimated) on State and local roads	Probable expenditures by State highway departments								Probable expenditures on roads and bridges by local authorities	Estimated road mileage to be built by State highway departments				Miles of State roads maintained by State highway department
		Total State road expenditures	Construction expenditures			Maintenance [1]		Interest payments, State highway and bridge bonds	Miscellaneous, including overhead		Total mileage	Earth, improved	Sand-clay, gravel, and macadam	Asphalt, concrete, and brick	
			Total roads and bridges	Roads	Bridges	Reconstruction	Road upkeep								
Alabama	$8,791,625	$5,291,625	$1,250,000	$1,050,000	$200,000		$2,000,000	$1,866,625	$175,000	$3,500,000	125	50	75		5,207
Arizona	8,600,000	6,100,000	4,350,000	4,050,000	300,000		1,260,000		550,000	2,500,000	360	105	250	5	2,635
Arkansas	10,855,300	8,555,300	(2)				1,500,000	[3] 6,980,300	75,000	2,300,000	(2)				7,500
California	69,600,000	36,400,000	23,000,000	20,000,000	3,000,000		6,000,000	2,800,000	4,600,000	33,200,000	520	50	60	410	7,103
Colorado	10,426,000	7,926,000	5,444,000	5,244,000	200,000		1,800,000	265,000	417,000	2,500,000	362	30	306	26	[4] 4,610
Connecticut	18,572,250	15,572,250	11,250,000	10,800,000	450,000		2,725,600	192,250	1,405,000	[5] 3,000,000	(6)				2,050
Delaware	4,328,000	2,948,000	2,324,000	2,124,000	200,000		363,000	116,000	205,000	1,380,000	100		25	75	960
Florida	10,139,000	8,139,000	4,107,501	3,142,901	964,600	$2,445,490	1,331,609		255,000	[5] 2,000,000	(6)				5,359
Georgia	20,500,000	16,500,000	13,250,000	12,750,000	500,000		2,250,000		1,000,000	[5] 4,000,000	850	200	50	600	7,138
Idaho	5,826,250	4,326,250	2,780,000	2,550,000	230,000		1,250,000	46,250	250,000	1,500,000	622	8	614		3,080
Illinois	79,080,000	51,080,000	37,500,000	35,500,000	2,000,000		5,250,000	5,830,000	2,500,000	28,000,000	1,000			1,000	11,000
Indiana	44,975,580	28,045,580	20,196,000	14,096,000	6,100,000		4,400,000		3,449,580	16,930,000	450	[5] 25	[5] 25	[5] 400	6,964
Iowa	40,298,000	20,298,000	11,000,000	9,500,000	1,500,000		3,110,000	[3] 4,340,000	1,848,000	20,000,000	1,225	350	675	200	7,260
Kansas	17,350,000	13,750,000	8,000,000	7,000,000	1,000,000	1,250,000	3,500,000		1,000,000	3,600,000	1,243	343	800	100	8,690
Kentucky	27,030,000	22,030,000	15,500,000	11,000,000	4,500,000		5,000,000	30,000	1,500,000	5,000,000	950	[5] 400	[5] 350	[5] 200	6,000
Louisiana	39,642,375	33,642,375	25,000,000	21,000,000	4,000,000		3,500,000	2,417,375	2,725,000	6,000,000	2,300	300	1,400	600	10,500
Maine	21,464,420	16,464,420	11,820,000	9,570,000	2,250,000		2,900,000	879,420	865,000	5,000,000	579		496	83	5,000
Maryland	13,458,316	10,958,316	6,046,075	3,790,542	2,255,533	2,802,241	2,110,000		(7)	[5] 2,500,000	290		90	290	3,446
Massachusetts	21,443,000	17,443,000	10,350,000	8,850,000	1,500,000	3,500,000	3,175,000	256,000	162,000	4,000,000	310		70	240	[8] 1,770
Michigan [9]	52,560,000	30,560,000	22,000,000	18,500,000	3,500,000		6,000,000	2,300,000	260,000	22,000,000	700	350	100	250	[10] 8,275
Minnesota	48,500,000	32,000,000	22,600,000	21,300,000	1,300,000	500,000	4,000,000	[3] 1,700,000	3,200,000	16,500,000	1,130	400	300	430	7,000
Mississippi	4,725,000	3,225,000	1,025,000	1,000,000	25,000		2,000,000		200,000	1,500,000	50	25		25	6,000
Missouri	52,906,300	40,906,300	31,106,300	30,778,000	328,300		4,500,000	3,800,000	1,500,000	12,000,000	4,517	2,105	2,034	378	9,000
Montana	11,950,000	6,950,000	5,000,000	4,000,000	1,000,000	250,000	1,000,000	100,000	600,000	5,000,000	900	450	450		3,500
Nebraska	18,350,000	10,350,000	6,600,000	6,000,000	600,000		3,250,000		500,000	8,000,000	1,025	100	800	125	7,400
Nevada	4,323,200	3,613,200	2,841,200	2,743,200	98,000		540,000	40,000	192,000	710,000	295	31	250	14	2,100
New Hampshire	8,867,000	6,867,000	3,700,000	3,500,000	200,000		2,000,000	592,000	575,000	2,000,000	150		100	50	2,500
New Jersey	67,187,243	45,568,601	28,275,457	15,100,457	13,175,000	1,000,000	2,500,000	7,371,475	6,421,669	21,618,642	128	30		98	1,400
New Mexico	5,856,312	5,656,312	3,180,000	2,615,000	565,000		1,200,000	536,312	740,000	200,000	312	11	295	6	5,700
New York	113,642,693	63,642,693	49,522,693	41,860,228	7,662,465		10,000,000	4,120,000	(7)	50,000,000	724	(11)	(11)	(11)	11,657
North Carolina	27,103,198	18,221,648	3,900,000	3,400,000	500,000		[12] 9,250,000	4,726,648	345,000	8,881,550	325		250	75	[13] 9,700
North Dakota	9,232,000	5,732,000	4,000,000	3,800,000	200,000		1,332,000		400,000	3,500,000	1,870	730	1,140		6,000
Ohio	63,500,000	33,500,000	20,500,000	17,000,000	3,500,000		13,000,000		(7)	30,000,000	955	35	150	[14] 770	11,743
Oklahoma	26,155,159	15,084,612	9,800,000	8,300,000	1,500,000	945,000	3,428,422		911,190	11,070,547	1,750	350	1,200	200	7,018
Oregon	19,000,000	11,000,000	5,500,000	5,000,000	500,000		3,000,000	1,340,000	1,160,000	8,000,000	325	150	170	5	[15] 3,800
Pennsylvania	118,830,000	68,830,000	34,800,000	34,800,000			22,500,000	3,830,000	7,700,000	50,000,000	2,700		2,600	100	34,074
Rhode Island	5,307,540	4,307,540	2,950,000	2,500,000	450,000		950,000	127,540	280,000	1,000,000	40			40	602
South Carolina [16]	13,550,000	8,550,000	5,000,000	[17] 5,000,000			2,000,000	1,100,000	450,000	5,000,000	(2)				5,907
South Dakota	10,230,000	6,230,000	3,530,000	3,330,000	200,000	75,000	2,125,000		500,000	4,000,000	718	290	400	28	5,250
Tennessee	14,550,000	10,050,000	3,600,000	3,000,000	600,000		3,500,000	2,300,000	650,000	4,500,000	180	20	10	150	7,500
Texas	60,000,000	45,050,000	32,000,000	28,000,000	4,000,000		12,000,000		1,050,000	14,950,000	2,500	1,325		1,175	18,868
Utah	5,675,000	4,475,000	2,750,000	2,150,000	600,000		1,300,000	325,000	100,000	1,200,000	454	20	427	7	4,092
Vermont [18]	5,837,500	5,087,500	2,650,000	2,500,000	150,000	500,000	1,500,000	262,500	175,000	750,000	160	30	85	45	1,013
Virginia	23,780,000	18,780,000	12,000,000	11,000,000	1,000,000		6,000,000	160,000	620,000	[5] 5,000,000	633	31	404	198	8,032
Washington	24,000,000	12,000,000	9,500,000	8,500,000	1,000,000	1,000,000	1,500,000			12,000,000	591	142	357	92	3,505
West Virginia	24,445,000	16,945,000	10,800,000	9,800,000	1,000,000		2,850,000	2,925,000	370,000	7,500,000	479	230	132	117	3,711
Wisconsin	33,760,000	17,760,000	11,900,000	11,000,000	900,000	906,000	4,094,000		860,000	16,000,000	523	73	104	346	10,186
Wyoming	6,634,000	5,134,000	3,600,000	3,350,000	250,000		950,000	184,000	400,000	1,500,000	675	210	465		3,125
Total	1,352,837,261	881,546,522	567,798,226	491,844,328	75,953,898	15,173,731	181,573,431	63,859,695	53,141,439	471,290,739	[19] 36,095	8,999	17,509	8,863	314,930

Remark: Above expenditures exclude State bond payments on principal.

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[1] When reconstruction is reported under "Maintenance" it is so shown; otherwise it is included with "Construction expenditures."

[2] No construction contemplated in 1932.

[3] Includes interest payments on local road bonds assumed by State.

[4] Excludes 261 miles of forest roads maintained by State.

[5] Estimate based on 1931 program.

[6] Mileage not reported.

[7] Included in construction and maintenance.

[8] Excludes 8,000 miles of town roads maintained by State.

[9] Excludes $4,000,000 for city street widening and $1,158,000 for maintenance of city streets under supervision of State Highway Department.

[10] Excludes 343 miles of city streets maintained by State.

[11] Types of mileage to be constructed not segregated.

[12] Includes $6,000,000 for county road maintenance by State highway department.

[13] Excludes 46,980 miles of county roads maintained by State highway department.

[14] Includes 350 miles to be widened and resurfaced.

[15] Excludes 2,000 miles of secondary highways maintained by State.

[16] Excludes $4,656,402 for reimbursements of counties for county bond payments.

[17] Amount payable on previous year's uncompleted work; no new State construction contemplated in 1932.

[18] Unofficial estimate reported by bureau engineer.

[19] Includes 724 miles in New York not included in mileage totals by types of road.

ROAD PUBLICATIONS of the BUREAU OF PUBLIC ROADS

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.

DEPARTMENT BULLETINS

*No. 136D . . Highway Bonds. 20 cents.
*No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.
*No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.
*No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.
*No. 660D . . Highway Cost Keeping. 10 cents.
No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
No. 1486D . . Highway Bridge Location.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys.
No. 265T . . Electrical Equipment on Movable Bridges.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

FEBRUARY 29, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$				$	$				$	
Alabama	2,405.5	1,516,104.91	736,156.40	62.5		62.5						5,432,927.22	Alabama
Arizona	1,153.8	2,777,948.05	1,903,876.08	167.7	57.4	215.1						1,538,171.97	Arizona
Arkansas	1,933.8	3,268,569.64	1,632,812.22	73.9	67.8	141.7	362,032.63	181,016.26	17.3		17.3	2,022,780.34	Arkansas
California	2,268.0	10,627,452.90	4,735,006.60	217.0	38.4	255.4	2,068,582.60	1,132,801.37	60.2	0.3	60.6	1,140,445.57	California
Colorado	1,525.4	6,327,506.01	2,727,704.45	219.4	66.2	285.6	352,906.90	172,218.97	14.4	3.1	17.5	2,031,759.87	Colorado
Connecticut	288.6	3,084,244.77	1,151,767.90	22.9		22.9						713,252.14	Connecticut
Delaware	361.6	302,770.00	161,386.00	16.2		15.2	416,243.25	208,121.62	18.8	2.5	21.3	288,388.31	Delaware
Florida	591.6	6,129,634.73	2,325,684.09	137.1		137.1	49,004.56	24,502.28	.1		.1	2,995,441.71	Florida
Georgia	3,042.5	6,035,595.78	2,805,155.19	144.0	150.1	294.1	765,828.90	382,913.43	37.9	8.4	46.3	1,858,089.10	Georgia
Idaho	1,486.3	1,726,513.35	998,918.80	85.9	80.3	166.2	507,309.62	306,240.89	28.5	8.8	37.3	1,273,887.54	Idaho
Illinois	2,631.6	23,516,320.34	10,992,229.09	694.2	28.7	722.9	3,380,155.14	1,357,398.18	110.7	12.8	123.6	2,620,147.69	Illinois
Indiana	1,779.4	7,932,830.68	3,890,986.20	210.0		210.0	4,368,682.66	2,042,480.46	163.9	7.3	171.2	1,015,757.76	Indiana
Iowa	3,390.6						2,294,391.13	1,073,198.25	88.6	38.4	127.0	1,726,606.95	Iowa
Kansas	3,655.6	2,772,849.63	1,295,773.35	155.3	16.3	170.6	3,217,254.92	1,573,735.01	215.4	5.4	220.8	1,653,758.28	Kansas
Kentucky	1,876.1	2,371,724.06	1,078,977.39	142.2	4.0	146.2	554,263.03	257,680.28	8.3	41.6	49.9	1,752,284.08	Kentucky
Louisiana	1,541.6	7,216,064.49	2,803,668.96	112.6	10.6	123.2	1,295,507.93	68,270.07	.6		.5	1,669,179.62	Louisiana
Maine	706.0	1,804,346.16	789,080.76	31.4		31.4	1,092,164.18	407,145.00	27.1		27.1	930,066.82	Maine
Maryland	777.3	269,787.15	63,776.89	6.2		6.2	327,551.20	163,775.59	14.4		14.4	753,906.63	Maryland
Massachusetts	814.8	7,521,359.24	3,084,670.77	66.8	.2	67.0						688,428.02	Massachusetts
Michigan	1,974.3	10,508,444.69	4,658,432.69	384.7	58.2	442.9	871,983.39	348,545.34	43.0	10.4	53.4	3,163,346.35	Michigan
Minnesota	4,236.4	2,777,868.36	945,585.45	41.1	71.2	112.3	7,435,259.66	2,406,185.76	164.4	182.6	346.9	299,654.50	Minnesota
Mississippi	1,792.4	4,006,906.67	1,964,657.82	169.4	78.0	247.4	103,524.13	51,762.05	9.5		9.5	5,390,115.02	Mississippi
Missouri	2,961.9	2,128,677.12	873,611.16	49.0	3.4	52.4	2,340,093.19	1,045,562.84	81.4	10.3	91.7	2,765,682.31	Missouri
Montana	2,659.9	4,714,534.97	2,650,928.65	406.6	17.6	423,2	403,394.86	224,368.61	61.5	.5	62.0	3,409,512.56	Montana
Nebraska	4,143.8	5,045,660.06	2,469,130.70	213.8	76.6	290.4	341,832.91	170,752.29	13.9		13.9	2,535,115.92	Nebraska
Nevada	1,352.3	1,188,334.90	916,071.55	40.1	73.6	113.7						1,387,101.79	Nevada
New Hampshire	417.2	676,612.12	265,632.44	13.4	2.5	15.9						647,810.24	New Hampshire
New Jersey	696.7	4,537,831.06	1,624,072.73	44.9	.6	45.5						1,604,603.13	New Jersey
New Mexico	2,166.3	2,839,017.62	1,646,283.94	135.7	24.3	160.0	390,818.65	245,489.48	14.1		14.1	1,285,729.48	New Mexico
New York	3,262.6	11,450,810.00	6,290,930.00	267.7		267.7	4,429,800.00	1,736,750.00	86.5		86.5	3,937,031.23	New York
North Carolina	2,222.0	1,323,877.49	640,966.07	69.7		69.7	144,243.14	72,121.56	8.0	4.0	12.0	3,873,930.55	North Carolina
North Dakota	5,117.4	1,828,691.83	918,958.57	160.4	171.9	332.3	1,327,226.48	679,129.08	149.7	259.5	409.2	2,097,184.05	North Dakota
Ohio	2,847.8	6,806,146.56	2,366,982.51	96.0	19.2	115.2	749,700.00	366,240.00	11.6		11.6	4,114,890.49	Ohio
Oklahoma	2,210.3	4,804,687.46	2,333,810.09	129.1	75.4	204.5	905,270.12	429,123.79	64.7		64.7	2,114,538.86	Oklahoma
Oregon	1,537.0	3,770,922.17	2,076,608.52	93.9	47.1	141.0	134,459.60	83,674.19	11.8		11.8	1,795,507.85	Oregon
Pennsylvania	3,012.1	4,193,848.20	1,917,622.88	46.8		45.8	993,951.13	453,156.32	32.0		32.0	4,705,730.74	Pennsylvania
Rhode Island	257.6	809,280.68	400,528.17	16.6		16.6	81,305.62	40,652.81		2.2	2.2	338,736.37	Rhode Island
South Carolina	1,996.3	2,679,138.08	1,148,707.78	63.0	43.6	106.6						1,405,597.62	South Carolina
South Dakota	4,035.8	3,167,197.01	1,748,253.46	185.9	176.3	361.2	475,473.18	318,254.13	81.9		81.9	1,136,687.89	South Dakota
Tennessee	1,666.8	796,986.37	394,765.99	24.6	4.4	29.0	531,028.30	249,959.65	14.1	6.4	20.5	3,830,871.62	Tennessee
Texas	7,574.6	13,699,494.07	6,196,407.60	716.2	162.3	878.5	2,384,760.40	1,084,444.04	166.4	54.8	221.2	6,506,492.19	Texas
Utah	1,218.5	690,137.33	482,342.89	60.9	.4	61.3	316,975.19	234,536.62	21.6	3.3	24.9	1,325,708.11	Utah
Vermont	339.1	104,832.58	52,345.06	4.3		4.3						572,077.12	Vermont
Virginia	1,861.1	2,551,746.36	1,222,025.16	134.3	16.3	150.6	325,632.43	162,816.21	28.1		28.1	2,013,258.14	Virginia
Washington	1,195.3	2,325,797.79	1,120,168.56	75.1		75.1	657,348.73	280,800.00	22.1	11.1	33.2	1,556,569.91	Washington
West Virginia	882.9	1,735,051.39	761,625.34	44.6	10.5	55.1	187,685.92	86,032.31	.7		.7	1,238,151.24	West Virginia
Wisconsin	2,673.6	2,818,682.42	1,075,507.70	131.1	7.8	138.9	110,272.34	50,000.00	1.1	3.5	4.6	2,699,689.10	Wisconsin
Wyoming	2,029.9	2,473,695.98	1,469,907.17	186.1	76.6	262.7	656,430.97	425,448.15	33.7	64.4	98.1	626,649.18	Wyoming
Hawaii	63.1	974,113.04	446,557.39	30.0		30.0	82,324.99	22,869.00	1.5		1.5	2,018,071.60	Hawaii
TOTALS	100,523.4	200,620,534.13	93,244,947.17	6,587.3	1,735.8	8,323.1	47,422,671.78	20,620,170.79	1,929.4	741.5	2,670.9	102,411,334.78	TOTALS

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 2 APRIL, 1932

A SCENIC HIGHWAY OF THE WEST

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 2 April, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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WHERE THE HIGHWAY DOLLAR GOES

A STUDY OF COST DISTRIBUTION IN THE CONSTRUCTION OF CONCRETE PAVEMENTS

Reported by J. L. HARRISON, Division of Management, U. S. Bureau of Public Roads

DURING the past few years, but more particularly since the beginning of the present business depression, there has been a good deal of reference to larger construction programs as a means of relieving unemployment. In the very nature of the case, an increase in activity in any line of construction work involves the employment of additional labor both on the job itself and in the preparation of the materials required in its execution.

All lines of construction work are valuable aids during seasons of unemployment but none is more effective as a means of providing widespread employment than highway construction, particularly of the higher types, involving as it does the direct employment of a considerable number of men on the job and the indirect employment of a far greater number, usually in the more thickly populated centers, in preparing materials, manufacturing equipment and supplies, and in transporting them.

The widespread influence of Federal-aid highway work is especially noteworthy. The Federal-aid road act, effective since 1916, provides that all highway funds must be apportioned to the States and be expended therein on the Federal system of highways. To date, as a result, the effect of Federal-aid highway expenditures has been felt in practically every county in the country.

In view of the foregoing, this article has been prepared, in which is set forth as definitely as available data will permit the extent to which expenditures for highway improvement provide employment. The considerable extent to which such expenditures contribute to the support of others than those directly and indirectly employed is not analyzed in this article.

It may be observed that the method of analysis followed in this article could be applied to any process of manufacture or to any activity involving the use of labor and salaried personnel. The results of such an analysis would show, in most cases, that the great bulk of expenditures are chargeable, either directly or indirectly, to the account of salaries and wages. It happens that the highway contracting business operates in general on a very narrow margin of profit; and this is also currently true of the activities indirectly connected with the construction of high-type roads, and concrete roads in particular—cement manufacture, the quarrying of aggregates, railway transportation, and coal mining. For this reason it is at least probable that the contribution to labor is higher in the case of road-building than in the case of many other industries. However, the chief advantage of increased highway construction during times of economic depression lies in the fact that it affords much direct employment and stimulates many lines of business activity at a time when such an impetus is greatly needed, while at the same time meeting the increasing demand for improved roads.

For the purpose of the present study, concrete pavement has been selected largely because it is widely used on the Federal-aid highway system, and because the effect of expenditures for pavements of this type is believed to be typical of the effect produced by expenditures on other high-type pavements as well. Furthermore, it is typical of the effect produced by expenditures for public works generally, which have a wider influence than might at first be supposed. The points at which the influence of different types of construction is applied differ, however. The erection of a steel bridge yields a different distribution of effect than is involved in the construction of a concrete pavement, and neither shows exactly the same distribution as the construction of a monumental building. But this is of little consequence, for though the application of effect is at different points, in all of these cases the general effect is much the same.

The discussion which follows shows that while about 80 to 90 per cent of the money expended for concrete pavement construction is ultimately paid out as salaries and wages, only about one-seventh or approximately 15 per cent is paid to those employed on the jobs where these pavements are being built.

Further analysis shows that about one-third of the cost of pavement is paid to producers of materials (sand, gravel, stone, cement, and steel) which are components of such construction. Another third is paid to the railroads and other transportation companies for transport service on account of the collection and delivery of these materials. A large part of the remaining cost is paid to the producers of equipment. In all of these fields the gross payments made by the contractor are further distributed by the producer or the manufacturer through his pay rolls and expense payments. Labor receives the major share of this distribution as the materials entering into highway construction are of little value in their original state. Values are created by the application of labor in manufacturing processes and in transportation.

Labor employed on the job in quarries and pits, cement mills, by railroads and other transportation agencies, in the steel mills, powder mills, blast furnaces, ore pits or deposits, by equipment and machinery manufacturers, and by producers and distributors of the materials and supplies used by the above agencies, receives some part of the dollar expended for highway construction. For the most obvious phases of such construction, i.e., laying the pavement, production of materials, and transportation of materials and equipment, about 15 per cent is paid to labor utilized on the job; about 12 per cent for labor in the quarries, mills, gravel and sand pits producing materials for the pavement; and about 14 per cent as salaries and wages to railroad and other transportation employees engaged in transport of materials to the job.

In addition to these operations, the equipment must be built; repairs and spare parts must be provided. Materials other than those cited above are required for equipment needs, all of which involve labor and additional transportation which again involves additional labor, with the final result that instead of the 40 per cent of cost allotted to labor employed on the most obvious phases of the work it appears that about three-quarters of the cost of the pavement is ultimately paid out as compensation for the work done in the processes, operations, and manufacture by which the multitude of raw materials are produced, manipulated, transported, and finally combined into the finished pavement. Thus, several times as much money is finally disbursed to labor in a widely diversified field of occupations as is paid to the labor actually engaged in laying the pavement.

Of the balance, which is made up of such items as profit, interest, rents, and charges for depletion, the major portion is reexpended as capital investment for new construction in some form, as for new factories or new buildings, where about the same proportion of the total expenditure is paid out in salaries and wages as is paid out in the highway field. When the salaries and wages resulting from such use of profits are added to the salaries and wages developed above, it will be found that about 90 per cent of all payments for the construction of concrete pavements is ultimately converted into salaries and wages.

The analysis is terminated at the point where further employment of the funds concerned takes the form of personal expenditures by the recipients of wages, profits, interest, rents, etc. This further turn-over of the money originally expended for highways undoubtedly acts as a stimulus to business in general; the present discussion, however, is concerned only with the share received by the labor and salaried personnel employed, directly or indirectly, in the construction of highways.

COST ITEMS ANALYZED IN DETAIL

It will be of interest to examine these expenditures in greater detail, and for this purpose the following primary separation of the cost of concrete pavements has been made.

1. The direct cost of laying concrete pavement (production expense).
    - *a.* Labor.
    - *b.* Aggregates.
    - *c.* Cement.
    - *d.* Steel.
    - *e.* Equipment.

These items cover the labor employed in connection with the various operations incident to laying the pavement, the cost of all materials of which the pavement is composed and the costs represented by the equipment used.

2. Expense incurred in connection with laying concrete pavement other than for production.
    - *f.* Installation of plant.
    - *g.* Bonds and insurance.

There are always certain preliminary expenses incident to this work which must be met, such as the cost of getting on to the job, the cost of developing a working organization, and the cost of bonds and insurance.

3. Job margin.
    - *h.* Overhead.
    - *i.* Financing.
    - *j.* Net profit.

Overhead, which includes central office salaries, rented quarters, the cost of bidding, etc., together with the cost of financing, must be paid out of job margin. After these and related expenses are satisfied the remainder is the net profit on whatever money is invested in the enterprise.

Based on 1929 prices in three typical States we have the first column of Table 1, which follows, as being reasonably representative of the first or primary distribution of cost of an average concrete paving job. These costs per square yard, when reduced to equivalent parts of pavement work done to the extent of $1,000, result in the distribution that is used in tracing the conversion of moneys paid for the construction of concrete pavement into payments for salaries and wages.

TABLE 1.—*Primary distribution of cost of average concrete paving job, based on 1929 prices in 3 typical States*

Item	Cents per square yard	Dollars per $1,000 expenditure
a. Labor	$0.26	$141
b. Aggregate	.60	324
c. Cement	.60	324
d. Steel	.05	27
e. Equipment	.18½	100
f. Getting on to job	.05	27
g. Bonds and insurance	.04	22
h. Job margin	.06½	35
	1.85	1,000

A distribution based on 1930 prices would differ somewhat from the above. A still greater divergence would be observed between a distribution based on current prices and the distribution given above. Indeed, such a distribution of the cost of concrete pavement as the one given above changes with every modification in the cost of labor or the cost of materials. It also changes as the average efficiency with which this work is handled changes, as well as with every improvement in methods and machines.

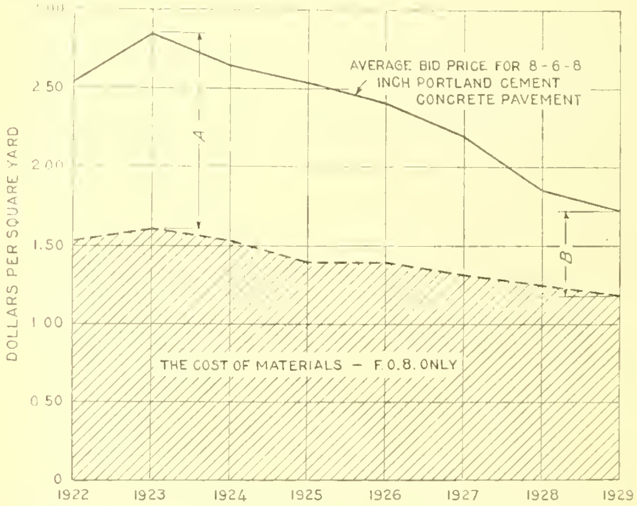

FIGURE 1.—TREND IN UNIT COST OF CONCRETE PAVEMENT FOR ONE STATE, DURING THE YEARS FROM 1922 TO 1929. THE MAXIMUM SPREAD, *A*, BETWEEN MATERIALS COST AND AVERAGE BID PRICE, IS $1.28; THE SPREAD IN 1929, *B*, IS NOT OVER $0.53

To show something of the nature of these changes Figure 1 has been included in this discussion. This figure is a graphical presentation of the trend in the cost of concrete pavement in one State, together with the trend in the cost of materials used in the concrete. It is to be observed that, while the cost of materials has decreased, the cost of the other elements has decreased quite as steadily and rather more rapidly. In short, such a distribution as that given in Table 1 is of momentary applicability only.

There would have been definite advantages in basing a discussion of this sort on current conditions rather than on those existing in 1929; but in an analysis of this sort much use must be made of figures regularly collected by the Bureau of the Census, and no figures collected by the Bureau of the Census on the subject of manufactures are available for any later date than the year 1929. Bureau of Internal Revenue figures have also been freely used and, when this analysis was made, these were not available for any later date than 1928.

DISTRIBUTION OF THE HIGHWAY DOLLAR IN THE MARKETS OF THE NATION

The primary distribution of costs in Table 1 shows the labor directly employed on the job as Item *a*. For the purpose of showing a résumé of the labor indirectly employed Table 2 is given below. This table is made up of a series of statements showing the accumulation of labor costs as one item after another is broken down to show the included labor cost. The analysis is discontinued at the point (section 7) where the remaining items other than salaries and wages are in the form of payments to owners, i. e., owners of the business, of property, or of money used in the various businesses involved. The extent to which these items, interest, rents, profits, and reserve, are ultimately expended in salaries and wages is discussed in the latter part of the report. Following Table 2 is a presentation of the supporting evidence on which these statements are based.

TABLE 2.—*Distribution of $1,000 paid for concrete highway, showing the approximate total amount which reaches labor in each of seven successive steps*

(1) The contractor's distribution of $1,000 expended (Table 1, column 2):

1. Labor	$141.00
2. Aggregates	324.00
3. Cement	324.00
4. Steel	27.00
5. Equipment	100.00
6. Plant installation	27.00
7. Bonding, etc	22.00
8. Gross profit	35.00
	1,000.00

(2) After distribution of contractor's charges (see Table 3):

1. Salaries and wages	$302.70
2. Freight	[1] 406.70
3. Materials and supplies	[1] 30.15
4. Fuel	35.50
5. Interest	14.10
6. Taxes	24.10
7. Depreciation and repairs	[1] 118.15
8. Depletion	10.50
9. Profit	48.10
10. For redistribution	10.00
	1,000.00

(3) After distribution of freight charges (see p. 30):

1. Salaries and wages	$477.70
2. Materials and supplies	70.55
3. Fuel	57.20
4. Interest, rents, etc	61.70
5. Taxes	49.70
6. Depreciation and repairs	171.65
7. Profit	91.00
8. Depletion	10.50
9. For redistribution	10.00
	1,000.00

(4) After distribution of fuel costs (see p. 30):

1. Salaries and wages	$516.00
2. Materials and supplies	77.20
3. Interest, rents, etc	63.75
4. Taxes	51.40
5. Depreciation and repairs	175.75
6. Profit	91.00
7. Depletion	14.90
8. For redistribution	10.00
	1,000.00

(5) After distribution of depreciation and cost of repairs (see p. 31):

1. Salaries and wages	$568.70
2. Materials and supplies	176.45
3. Interest, rents, etc	65.50
4. Taxes	55.80
5. Depletion	14.90
6. Profit	108.65
7. For redistribution	10.00
	1,000.00

(6) After distribution of cost of materials and supplies (see p. 31):

1. Salaries and wages	$730.90
2. Interest, rents, etc	73.95
3. Taxes	[2] 39.45
4. Depletion	17.95
5. Profit	127.75
6. For redistribution	10.00
	1,000.00

(7) After distribution of taxes and $10 for redistribution (see p. 31):

1. Salaries and wages	$771.50
2. Interest, rents, etc	81.30
3. Profit	129.05
4. Reserve for depletion	18.15
	1,000.00

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[1] These items adjusted for freight charges assignable to materials, supplies, replacements, and repairs.

[2] This item adjusted for amount transferred to materials and supplies.

Item a. Labor.—Table 1 indicates that 14.1 per cent of the cost of concrete paving is used directly in the payment of salaries and wages. In the production of concrete pavements labor is used directly in preparing the subgrade, in handling materials, in hauling them to the mixer, in operating equipment, and in taking care of the curing of the pavement, as well as for the purpose of accomplishing the numerous other details incident to this work. The general practice is to organize a paving crew which thereafter is of fairly uniform size and therefore of fairly uniform cost per working day. The labor element of cost listed above in the second column of Table 1 is the average cost of working this crew per $1,000 worth of Portland cement concrete pavement completed. The actual labor cost varies somewhat, the minimum being around $115 and the maximum around $175 with an average that appears to be not far from $141 of each $1,000 spent in this field.

This is not the whole cost of labor used directly on paving work, for, as will be developed in subsequent paragraphs, labor is used in getting on to the job and in preparing plant and equipment for the work of production that is to follow, and some expenditure for clerk hire, salaries, etc., is involved in overhead. As that part of the cost of pavement which is covered under the item labor is a matter of payments to labor used on the job, a further analysis of these payments is not required. The purpose here is to indicate the amount of the cost of pavements of this type which, in one way or another, is paid out as salaries and wages.

Item b. Aggregates.—Of the total cost of concrete pavement an average of about $324 per $1,000 spent is paid for aggregates (sand and gravel, stone, and slag) f. o. b. cars at the contractor's materials yard. This price includes two items: Freight charges and the cost of materials f. o. b. cars at points of origin—that is, at the quarries, gravel pits, sand pits, slag piles, etc., where these materials are produced. The data available are not in sufficient detail to indicate an exact division of cost between these two fields. In general, freight charges on aggregates run between 50 cents and $1.25 a ton, the average charge reported by several shippers being in the neighborhood of $1 a ton. This can not be far from correct as statistics published by the Interstate Commerce Commission (Freight Commodity Statistics, year ending December 31, 1929), show that during the year 1929 there were 81,408,069 tons of gravel and sand delivered to the railroads for shipment as revenue freight, and that the gross revenue from this source was $69,420,839, or about 85 cents per ton. Crushed, broken, or ground stone produced 33,529,078 tons of revenue freight from which the revenue was $29,860,479, or a little over 89 cents per ton.

As a general rule quarries, gravel pits, and sand pits are located with their major markets, the larger cities, in mind. When the average freight charges are considered in the light of this fact, the average freight charge on materials sent to highway projects—about $1 a ton as given by the larger producers—does not appear to be estimated too high.

The Bureau of Mines (Sand and Gravel in 1928 and Sand and Gravel Production in 1929) gives the production and the value of paving sand and paving gravel produced in the United States as follows (average values as shown are computed from the figures given by the Bureau of Mines):

		Tons	Value	Value per ton
Paving sand:	1927	35, 606, 622	$17, 767, 491	$0. 50
	1928	35, 244, 544	17, 305, 750	. 49
	1929	40, 801, 991	21, 131, 731	. 52
Paving gravel:	1927	44, 891, 975	29, 887, 365	. 67
	1928	49, 088, 786	30, 697, 993	. 63
	1929	60, 029, 164	38, 695, 207	. 64

Similar figures on broken stone for concrete and road metal as released by the Bureau of Mines (Stone in 1928 and Production of Stone in 1929) are as follows:

		Quantity	Value	Value per ton
Broken stone:	1927	78, 544, 210	$84, 177, 237	$1. 07
	1928	74, 384, 490	81, 041, 349	1. 09
	1929	76, 174, 770	80, 685, 493	1. 06

If the average freight charges on sand, gravel, and broken stone at $1 a ton are combined with the average sand and gravel prices given above, the following results:

	Sand	Gravel	Broken stone
Average values per ton	$0. 52	$0. 64	$1. 06
Freight charges per tou	1. 00	1. 00	1. 00
Value as delivered on the job	1. 52	1. 64	2. 06
Approximate price per cubic foot delivered	. 07½	. 08½	. 10½
Percentage chargeable to freight	66	61	49

These prices per cubic foot of aggregate delivered on the job are substantially in harmony with the average cost of aggregate given above—about 60 cents per square yard of pavement laid. Therefore, they are used as reasonably accurate and as indicating that the charges for freight absorb about 60 per cent of the cost of aggregate delivered on the job.

Meticulous accuracy would require a somewhat closer division of costs than this and a somewhat wider distribution of transportation charges, for some of the aggregate used in pavement construction is moved from points of origin to the site of the work in trucks, and a small fraction is barged. However, no exact data on the amounts so handled are available. The great bulk of these materials move by rail and the division of cost f. o. b. contractors' yards, though an approximation, is as accurate as available figures will produce and quite accurate enough to yield significant results.

Concerning the distribution of the cost of sand and gravel f. o. b. cars at points of origin, the Bureau of the Census has released preliminary data under date of January 28, 1931. This release is based on the production during 1929 of 165,526,075 tons of sand and gravel in plants producing more than 25,000 tons annually. For this same period the Bureau of Mines reported a total production of 99,253,054 tons of sand and 123,318,851 tons of gravel. The average vlaue of the total production in both cases is about 50 cents a ton. Therefore, while it would seem that the figures collected by the Bureau of the Census omit a considerable tonnage of material produced by the smaller companies, it also is apparent that the average value of the product has not been disturbed by this situation.

The data as to the sand and gravel industry released by the Bureau of the Census includes the following information:

	Amount	Per cent
Value of product, f. o. b. plant	$100, 016, 527	100. 0
Salaries and wages, including coutract work	33, 470, 169	33. 5
Supplies	11, 666, 227	11. 7
Fuel, including purchased electric power	8, 573, 371	8. 6
Equipment purchased during year	6, 757, 694	6. 8

The census of mines and quarries for the year 1919 gives the following data as to quarrying costs:

	All rocks	Limestone only	Per ceut [1]	
Capital	$148, 759, 533	$82, 124, 367		
Salaries	7, 168, 303	3, 726, 593	7. 0	53. 5
Wages	45, 534, 798	23, 926, 332	45. 2	
Contract work	995, 976	665, 557	1. 3	
Materials and supplies	18, 441, 459	10, 968, 220	20. 8	
Fuel	5, 267, 846	2, 897, 432	5. 5	
Purchased power	2, 213, 459	1, 278, 958	2. 41	
Royalties and rents	1, 381, 290	667, 751	1. 35	
Taxes	2, 088, 170	1, 119, 861	2. 01	
Total expense	83, 091, 301	45, 250, 704	85. 6	
Total value, all products	101, 684, 919	52, 943, 924		
Value of stone produced	100, 423, 476	51, 967, 290		
Value of other products	1, 261, 443	976, 634		
Stone produced, tons	67, 884, 000	49, 715, 000		

[1] Percentage which each item in the column headed "Limestone only" bears to total value all products, $52,943,924.

Since these figures were collected there has been some change in quarry practices but, on the whole, rather less than in most production fields, for the general methods followed in quarrying, handling, crushing, and loading stone are very much the same to-day as they were 10 years ago. Therefore, though production has risen a good deal (being given by the Bureau of Mines as 136,345,130 tons in 1927, 133,869,510 tons in 1928, and 141,109,580 tons in 1929) the cost distribution has changed only a little. This may be seen from the fact that the Bureau of Mines shows 22,967,579 man-days worked during 1929. (Quarry Accident Statistics.) Reports issued by the Bureau of Labor Statistics (Trend of Employment and Labor Turnover, October, 1930, p. 2, and Monthly Labor Review, April, 1930, p. 158) indicate that the average weekly wage, full time worked, in the quarries operated by cement companies was a little under $30 a week. Assuming that the wages paid in independent quarries were about what they were in the quarries owned by the cement companies, labor cost in 1929 could not have been much, if any, over $110,000,000 and probably was about $103,000,000. The total value of the stone marketed was $202,692,762, so that the labor fraction probably exceeded 50 per cent of the value of the stone f. o. b. cars at the quarries.

In the statistics issued by the Bureau of Mines, nondimension stone is charged with 14,987,172 man-days of labor, which, at the figures given above, must have cost somewhere between $68,000,000 and $75,000,000. Nondimension stone is assumed to include:

	Tons	Value
Crushed stone	92, 721, 260	$94, 387, 878
Furnace flux	24, 393, 500	18, 034, 910
Refractory stone	1, 558, 200	1, 807, 324
Agricultural limestone	2, 654, 580	3, 764, 775
Manufacturing industries	7, 430, 749	6, 361, 201
Rubble	907, 810	1, 324, 681
Riprap	4, 212, 990	5, 030, 743
	133, 879, 089	130, 711, 512

From this it would also appear that the labor fraction in the cost of crushed stone is still somewhere between 50 per cent and 60 per cent of the total cost; in other words, that the 1919 cost distribution given above is not inapplicable to crushed stone production under current conditions.

Of the aggregates used in producing concrete pavements about 35 per cent is sand. There are no very reliable data as to the amount of gravel used as compared to the amount of broken stone used. However, it will be assumed that 45 per cent is gravel and 20 per cent broken stone, a distribution believed to be reasonably accurate.

If the figures for gravel and sand production costs and the distribution of the production cost of broken stone as given above are combined on this basis, it would appear that the following is a reasonable distribution for the aggregate field taken as a whole.

	Per cent
Salaries and wages	38
Supplies	14
power	8½

These figures account for about 60 per cent of the value of the aggregates purchased; the balance, nearly 40 per cent, is unexplained in this statement. In the more complete distribution of this item the latest reports of the Bureau of Internal Revenue (Statistics of Income for 1928) give the following information which is at least reasonably applicable.

Item 8. Mining and quarrying not elsewhere specified—Lessors and holders.—(Note.—This item includes corporations, but not individuals, operating quarries, gravel pits and sand pits, as well as a fraction of other operations in the mining and quarrying field not otherwise listed.)

Gross sales—	
Corporations reporting net income	$60, 284, 447
Corporations reporting no net income	46, 917, 270
Total	107, 201, 717
Net income or deficit—	
Corporations reporting net income	22, 504, 402
Corporations reporting no net income	—16, 817, 324
Total net income	5, 687, 078
Net income, per cent	5. 31

The figures published by the Bureau of Internal Revenue do not include a further distribution of Item 8, but for the mining and quarrying industry as a whole, the following figures are given:

	Per cent
Cost of goods sold	59. 79
Compensation of officers	1. 54
Interest paid	2. 76
Taxes paid, other than income	2. 22
Bad debts	. 32
Depreciation	5. 74
Depletion	5. 89
Miscellaneous deductions	16. 36
Net profit	5. 38
	100. 00

Combining these figures with those developed above, it would appear that the following is reasonably representative of the distribution of costs in the production of aggregate:

	Per cent
Salaries and wages	38. 0
Supplies	14. 0
Fuel, including purchased power	8. 5
Interest paid	2. 8
Taxes, other than income	2. 2
Bad debts	. 3
Depreciation	5. 7
Depletion	5. 9
Miscellaneous expense	17. 3
Profit (5.3 per cent less Federal income tax)	3. 1
Income tax	2. 2
	100. 0

While miscellaneous expense no doubt contains a variety of items, in this particular field it is so largely a matter of repairs and replacements of equipment that it will hereafter be so treated.

Applying these figures to that part of each $1,000 spent for aggregates, the following table results:

Proportion of each $1,000 spent for concrete roads generated by expenditures for materials	$324
Freight	194
Salaries and wages	50
Supplies	18
Fuel, including purchased power	11
Interest	4
Taxes, including income tax	6
Bad debts, less than 50 cents	[1]
Depreciation and depletion	15
Repairs and replacements	22
Profit	4
	324

Item c. Cement.—The cost of cement absorbs an average of about $324 out of every $1,000 spent on concrete pavement. The average cost of cement is fairly accurately indicated by the fact that during the fiscal year ended June 30, 1929, vouchers for Federal-aid payments included 4,278,485 barrels of cement listed separately. The cost of this cement, including freight, was $8,855,676 or an average cost of about $2.07 per barrel. During 1929 there were 25,008,203 tons of cement delivered to the railroads for shipment and freight charges to the amount of $67,083,562 were collected. This indicates an average freight charge of about 51 cents per barrel and an average value for the cement purchased separately for use on Federal-aid highway construction of about $1.56 per barrel. The figures published by the Bureau of Mines give the average mill price of cement as $1.62 during 1927; as $1.57 during 1928; and as $1.48 during 1929. The Bureau of the Census figures indicate an average value for 1929 of $1.53. This figure is based on a production of 169,868,322 barrels of Portland cement the value of which is given as $260,428,797. On the basis of 51 cents for freight and $1.48 as the 1929 price of cement, it would appear that about three-fourths of the cost of cement is a charge for cement at the mills, and that about one-fourth is a charge for freight; or, that of the $324 per $1,000 spent on pavements about $81 is paid for freight and about $243 is paid for cement f. o. b. cars at the cement mills.

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[1] Dropped.

As in other cases where freight is involved it is, of course, obvious that the railroads do not transport all of the materials shipped. Some of it moves in barges and quite a little moves in trucks; however, by far the larger part of the cement used in constructing highways moves by rail, so that for simplicity in analysis it will be assumed that it all does.

The census of manufacturers gives the following figures for the cement industry for 1927 shown in the first column and for 1929 the more limited information shown in the second column (Summary by Industries, 1929).

	1927	1929	Value per barrel	
			1927	1929
Value of Portland cement produced	$287, 431, 268	$260, 428, 797	$1. 66	$1. 53
Salaries [1]	16, 331, 586		.093	
Wages [1]	53, 110, 475	47, 872, 091	.303	.278
Materials and supplies [1]	59, 495, 154	92, 599, 816	.339	
Fuel and power [1]	60, 516, 871		.345	

[1] These figures include natural and puzzolan cement, valued at less than 2 per cent of total value of all cement. They are obtained by dividing the costs shown in columns 1 and 2 by the corresponding production including natural and puzzolan cement (1927, 175,330,381 barrels; 1929, 172,052,493 barrels).

During the hearings before the Committee on Ways and Means (House of Representatives, 70th Cong.), representatives of the cement companies filed various documents dealing with the cost of producing cement. Table 2 from page 8,344 of volume 15 of these hearings, which is based on data reported by 101 plants, gives the information shown in the first column below. These figures are for the year 1927. The second column gives similar figures compiled from reports from 122 mills for 1929 which were presented before the United States Tariff Commission in connection with a recent hearing on the duties on cement.

	1927 cost in cents per barrel	1929 cost in cents per barrel
Raw materials quarried and purchased, including labor	20. 12	20. 11
Fuel	24. 30	21. 86
Manufacturing labor	14. 21	11. 49
Packing and shipping labor	4. 07	
Power, including labor	13. 19	11. 62
Other manufacturing expense, including labor	14. 63	14. 81
Packing and shipping, exclusive of labor	5. 44	[1] 9. 58
General and administrative expense	21. 64	13. 03
Selling expense	13. 85	14. 61
Depreciation on buildings and equipment		11. 77
Inventory adjustment		. 26
Interest on investment (computed at 6 per cent)	19. 14	19. 50
	150. 59	148. 64

[1] Includes labor.

Another table given in the report of hearings before the Committee on Ways and Means (p. 8345), based on reports from a limited number of plants but believed to be reasonably representative, gives the 1927 labor costs of the following items not separately reported above as follows:

	Number of plants reporting	Labor cost in cents per barrel
Quarrying	82	5. 40
Power	10	2. 69
Repairs	12	2. 73
Laboratory	8	1. 31

The data presented to the United States Tariff Commission subdivides selling expense during 1929 as follows:

	Cents per barrel
Salesmen's salaries and expenses	6. 98
Commissions	. 31
Other selling expense	7. 32
	14. 61

The figures given above indicate that the cost of raw materials did not change materially from 1927 to 1929. The cost of power dropped a little, presumably on account of changes in the cost of coal, and in the general efficiency of operation, which evidently was somewhat improved; for the Bureau of Mines reports 135.8 pounds of coal used per barrel of cement produced in the mills that used coal as fuel in 1927, against a consumption of 132.3 pounds per barrel in 1929. The cost of packing and shipping changed very little. Presumably the labor used in making repairs and the salaries and wages paid in the laboratories were equally stable.

In view of these facts it would appear that, exclusive of salaries and wages, which are a part of administrative expense or a part of selling expense, the cost of the labor used in producing cement during 1929 is indicated with reasonable accuracy by the following table:

Labor costs per barrel of cement produced

	Cents per barrel
Manufacturing labor	11. 49
Packing and shipping	4. 07
Quarrying	5. 40
Power	2. 69
Repairs	2. 73
Laboratory	1. 31
	27. 69

This figure is in substantial agreement with the figure given by the Bureau of the Census—27.9 cents.

Financial reports of cement companies controlling about 44 per cent of the national production capacity indicate a bonded indebtedness in 1930 of $41,735,500, which suggests a total outstanding bonded indebtedness for the industry as a whole of about $95,000,000. Interest charges on this sum at 5 per cent amount to $4,750,000 and suggest an average interest change of about 2.8 cents per barrel of cement produced.

Depreciation is given above as 11.77 cents per barrel of cement produced. Financial reports of cement companies controlling about 44 per cent of the total production capacity show charges in 1930 of a little over $10,700,000 for depreciation and depletion, which suggests a total charge for all companies of about $24,400,000. As this charge does not change a great deal from year to year, it would appear that depreciation and depletion amount to about 14.3 cents per barrel. The difference, 2.53 cents per barrel, is, at least in large part, chargeable to depletion. However, this amount is somewhat more than an examination of various financial reports would indicate to be ordinarily allowed for depletion. For this reason depreciation and depletion are listed in the redistribution of cement charges as having the following values:

Depreciation	$20. 50
Depletion	3. 00

For the year 1927 salaries as distinct from wages were reported by the Bureau of the Census as $16,331,586. Business conditions in 1929 were such as to make it probable that this figure is reasonably representative of costs during the latter year, which would suggest a charge of about 9.5 cents for salaries for the year 1929.

Taxes are not known with any great degree of exactness. The figures given by the Bureau of Internal Revenue (Statistics of Income for 1928) indicate that the taxes, including Federal income tax paid by the stone, clay, and glass industries, of which the cement business is roughly a quarter, amounted to a little over 2 per cent of the gross income. This would suggest a charge of about 3 cents a barrel on the cement produced.

On the basis of the figures given above, the cost of producing cement may be recast in the form used in this discussion, as follows:

	Cents per barrel
Salaries and wages	37. 19
Supplies (raw materials, less labor; shipping expense, less labor; and other manufacturing expense, less labor)	32. 30
Fuel and power (less labor)	30. 79
Interest	2. 80
Taxes (unknown)	3. 00
Depreciation	14. 30
Miscellaneous expense	8. 76
	129. 14
Profit on the basis of Bureau of Mines average value	18. 86
Bureau of Mines average value at mills	148. 00

The item miscellaneous expense was obtained by adding the items of general and administrative expense, selling expense, and inventory adjustment, as given in the cost per barrel figures submitted at the hearings of the Committee on Ways and Means (p. 26); and deducting items chargeable to those accounts which have been otherwise cared for in this analysis. On this basis we have—

		Cents per barrel
General and administrative expense		13. 03
Selling expense		14. 61
Inventory adjustment		. 26
Total		27. 90
Less—		
Salaries	9. 50	
Laboratory labor	1. 31	
Taxes	3. 00	
Interest	2. 80	
Depreciation in excess of that shown in previous table (p. 26)	2. 53	
		19. 14
Miscellaneous expense		8. 76

The largest items appearing in the materials and supplies account beside raw materials are estimated for the year 1927 as follows. (See testimony before the Committee on Ways and Means, 70th Cong., vol. 15, pp. 8295 to 8389.)

Gysum	$5, 822, 498
Explosives	3, 467, 496
Bags	2, 201, 298
Mill supplies and machinery	21, 695, 496
	33, 186, 788

Of the above, explosives are of course a part of the charge for raw materials. If these are deducted the other materials listed generate a charge of about 17.2 cents per barrel of cement, which, with the reported cost of raw materials (less labor) at 14.71 cents, amounts to about 31.9 cents or about 2 cents less than the total the Bureau of the Census allocated to the item of materials and supplies (1927). This would seem to be a reasonably accurate cross check though it is probable that some of the cost of mill supplies and machinery as reported by the mills was generated by quarry operation and therefore may appear in the cost of raw materials.

Fuel and power are reported by the census (1927) as having cost $60,516,871, or 34.5 cents per barrel. The Bureau of the Census figures for 1929 do not segregate this item. The producers report—

	1927 (cents per barrel)	1929 (cents per barrel)
Fuel	24. 30	21. 86
Power (including labor amounting to 2.69 cents, 13.19 cents for 1927, and 11.62 cents for 1929)	10. 50	8. 93
	34. 80	30. 79

The Bureau of Mines (Cement in 1929, p. 409) gives the consumption of fuel in the production of Portland cement as follows:

Coal	tons	9, 340, 434
Oil	barrels	2, 533, 911
Natural gas	cubic feet	41, 643, 433, 045

Coal consumption was at the rate of 132.3 pounds per barrel of cement produced in the mills using coal as fuel, which would indicate that had coal been used exclusively the consumption would have been about 11,350,000 tons.

For 1929 the cost of fuel and power as reported by the cement companies, less 2.69 cents for labor, amounts to 30.79 cents per barrel or to something over $53,000,000. If for simplicity of analysis it is assumed that only coal was used for fuel, this would indicate a price of about $4.50 per ton delivered at the cement mills. Actually, the producers estimated the 1927 cost of coal as $42,330,074. The Bureau of Mines gives the amount of coal used in producing cement during 1927 as 9,990-531 short tons. This would indicate a price of a little less than $4.25 per ton, which must be very close to the correct figure, for the Bureau of Mines gives the average 1927 price of coal at the mines as $1.99. The average freight rate on bituminous coal is about $2.25.

The apparent cost of $4.50 noted above would seem to indicate either that the effective cost of oil and natural gas is a little higher than the cost of coal, or more probably that the total cost of fuel and power includes minor charges for oil, grease, etc. However, as the amount of charges of this nature is unknown, and as reasonable accuracy is preserved if the whole cost of fuel and power is assumed to be an expenditure for coal, further analysis will be on this basis. It would then appear that about 55 per cent of the cost of fuel and power, or about 16.9 cents per barrel of cement, is a charge for freight and that the balance, or substantially 13.9 cents, is a charge for the cost of fuel at the mines.

If the cost of mill supplies and machinery as given above for the year 1927 is divided by the production for that year, the resulting cost per barrel of cement produced is so close to the figure given for other manufacturing expense less the labor cost of making repairs as to suggest that other manufacturing expense is largely, if not entirely, a charge for repair and replacement parts. On this account the item supplies has been divided, repairs being charged at 12.08 cents per barrel and supplies and miscellaneous materials at 20.22 cents per barrel.

If the facts and the deductions made above are reduced to a statement of the number of dollars out of each $1,000 spent on concrete road construction each of these various items generates, the following table results:

Total value of cement used per $1,000 expended	$324.00
Freight	81.00
Cement at the mill	243.00
Salaries and wages	61.00
Repairs	20.00
Supplies and materials	33.00
Fuel	22.50
Freight	28.00
Interest	4.50
Taxes	5.00
Depreciation	20.50
Depletion	3.00
Miscellaneous expense	[2] 14.50
Profit	31.00
	243.00

The distribution of the cost of cement is now in a form in which the items that require further analysis can be transferred to accumulations of other similar items. However, for a few of these items a little further comment may be appropriate. Repairs are, of course, an equipment charge and are so transferred. Depreciation also is an equipment charge. Both apply to a wide variety of machinery and equipment which, in general, requires constant expensive repairs in spite of which it depreciates rather rapidly.

Miscellaneous expense, which is largely selling and administrative expense, is composed of a long list of items which it is impracticable to separate. There is some travel expense which includes railroad fares and hotel expense. Probably there is some expense for office rentals. There certainly is a charge for association dues. Advertising is an element in this expense. Office supplies, advertising, folders, booklets, etc., generate some of it. As no information concerning the amount of these expenses is available and as travel expense generates at least a considerable part of them, and arbitrary assignment of about one-third of this expense to railroad expense has been made and the balance will be treated as miscellaneous materials, which much of it is though some of it is not.

Item d. Steel.—Steel, principally reinforcing steel but also including longitudinal joints, dowel steel, chairs, etc., is a highly variable element in the cost of concrete pavements. In the States on which the primary distribution of cost used in this discussion is based, the average use of steel is somewhat less than it is for the country as a whole. However, this is not of great importance here, for increasing the amount of steel would not change the general nature of the distribution of the cost of concrete pavements, though it would somewhat alter the proportions allocated to the various primary subdivisions.

An analysis similar to that used in distributing the cost of cement results in the distribution of that part of each $1,000 spent on concrete pavement which in this discussion is allocated to the purchase of steel, as follows:

Salaries and wages	$6.00
Coal	1.50
Freight (of which $3 is freight on the finished product)	6.00
Equipment, repairs, depreciation	2.50
Materials	8.00
Interest	0.40
Taxes	0.90
Administration and sales expense [3] (railroad expense 20 cents; supplies 50 cents)	0.70
Profit	1.00
	27.00

Item e. Equipment.—The cost of equipment—$100 out of each $1,000 spent on concrete pavements—represents equipment costs of three kinds: Depreciation, repairs, and operating expense. The latter, under the subdivision of costs used in this discussion, is largely a matter of fuel, lubricating oil, and minor incidentals. The labor used in operating equipment has been included with other job labor under Item *a*.

The size and the amount of the equipment used in laying concrete pavement, except the amount of hauling equipment, is rather uniform from job to job. Moreover, the specifications under which concrete pavements are laid, though not altogether uniform, are sufficiently so to generate a fairly constant rate of depreciation for each kind of equipment used. (See Depreciation Studies and Bulletin F, Bureau of Internal Revenue.) Field studies made by the Bureau of Public Roads indicate that the average value of the equipment other than hauling equipment used on concrete paving jobs is about $50,000. The value of the hauling equipment varies widely, partly on account of variations in the type of hauling equipment used and partly on account of wide variations in the distance materials must be moved. In occasional instances not more than three or four 1½-ton trucks are required. Where the haul distance is from 15 to 20 miles as many as 20 heavy trucks, worth well over $100,000, are required.

Under such conditions the average of even a considerable number of values obtained from field studies is not as certainly accurate as it appears to be in the matter of other job equipment the value of which is more nearly constant from job to job, but this average value, about $25,000, is at least approximately correct. The details of a reduction of these conditions to a charge for depreciation and a charge for repairs do not require repetition here. Neither do the details of other analyses.

In the nature of the case, repair costs are at least largely a matter of the cost of spare parts and the transportation charges on them. Little extra labor is involved since most repairs are made by the men who operate the machines and their time is included in the cost of job labor (Item *a*). In theory, depreciation charges may accumulate as cash; in practice, they are expanded for renewals about as fast as they accrue. Both repairs and depreciation are therefore equipment charges. The analysis of these charges is not repeated here but the results of these analyses are shown in the following table:

Distribution of the amount allocated as a charge for equipment	$100.00
Primary distribution:	
Repairs and depreciation	64.00
Gasoline and oil	36.00

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[2] Railroad expense, $4.50; miscellaneous supplies, $10.

[3] As in the case of a similar item in the distribution of the charges generated in making and marketing cement, this charge covers a multitude of minor items of which, as travel expense is of considerable importance, about a third is allocated as railroad expense and the balance as supplies, which in fact much of it is.

Secondary distribution (gasoline and oil):

Freight	$4.50
Taxes	6.80
Wages (tank wagon)	4.00
Repairs and depreciation (tank wagon)	4.00
Profit	.20
Wholesale cost of gasoline	16.50
	36.00

Third distribution (refining cost of gasoline):

Salaries and wages	1.20
Materials and supplies	10.65
Fuel	.50
Repairs	1.85
Interest	.20
Taxes	.40
Depreciation	.80
Profit	.90
	16.50

Combined totals:

Salaries and wages	5.20
Freight	4.50
Taxes	7.20
Repairs and depreciation	70.65
Materials and supplies	10.65
Fuel	.50
Interest	.20
Profit	1.10
	100.00

Item f. Plant installation.—The item of getting on to the job, amounting to about $27 out of every $1,000 spent on concrete pavement, covers the cost of sending equipment to the job (usually a freight charge), of erecting it, organizing, preparing to start construction, and placing equipment in storage ready for reshipment. The amount of exact information that is available regarding the cost of these various operations is limited, but such as has been collected indicates that about half of this item is a freight charge and about half a labor charge. Distributed on this basis, this item reduces to—

Labor	$13.50
Freight	13.50

Item g. Bonds and insurance.—The reports of the Treasury Department for 1929 as to companies which issue surety bonds give the following data:

Premiums collected, a little over		$57,310,000
Losses and claims	per cent	43.5
Commissions and brokerage	do	26.3
Other expense (largely general overhead)	per cent	23.1
Balance (reserves)	do	7.1

Bonds for highway purposes provided about one-fifth of the total premiums collected. Of the charges listed above, losses and claims are, in effect, payments toward construction and should therefore be distributed just about as original payments are distributed. Commissions and brokerage payments are, in effect, salaries and wages paid those who obtain business for the surety companies. Other expenses are so largely salaries and wages paid central office employees that they will be so considered. Actually some other charges—as rents, office supplies, advertising, etc.—are included in this item but the details are not of record.

While by far the larger part of the charge for bonds and insurance is a charge for surety bonds, a fraction is for workmen's compensation, which, in the very nature of the case, generates payments to labor; and a little is for liability and other forms of insurance. On the whole, however, a reasonably accurate view of this matter is had if of the $22 paid out of each $1,000 spent on concrete road construction for bonds and insurance, $11 is charged to salaries and wages, $10 is charged back for redistribution with other primary costs, and $1 is considered as profit. Actually, most of this $1 goes to reserves; but from the standpoint used in this discussion, reserves being the property of the concerns owning them, it has the general nature of profit. This item as fractioned above becomes—

Salaries and wages	$11.00
Redistribution	10.00
Profit	1.00
	22.00

Item h. Profit.—Profit, as the term is used in this discussion, is job profit. Out of it there must be paid such items as administrative salaries, head office salaries, financing charges, office expenses, the balance, if any, being net profit. As to profit in this general field, which with several other lines of construction activity is covered by serial No. 54 in the classification established by the Bureau of Internal Revenue (Statistics of Income for 1928, p. 329) the following data appear:

Gross income:	
Corporations reporting net income	$1,138,033,597
Corporations reporting no net income	375,245,267
Total	1,513,278,864
Net income or deficit:	
Corporations reporting net income	69,297,579
Corporations reporting no net income	−32,728,384
Total net income	36,569,195

This would indicate an average net profit of a little more than 2 per cent on the volume of business reported. It is to be observed that these data cover only the corporations engaged in construction work. More than half of the total volume of all construction work is handled by individuals and by partnerships, generally only the larger and stronger concerns being incorporated. On this account it would appear to be reasonable to assume that the average net profit of corporations, partnerships, and individuals handling construction work is a good deal less than the average obtained by the corporations—possibly rather less than 1 per cent. This suggests that of the $35 set up as job profit per $1,000 spent on concrete pavements, about $10 may be set aside as a conservative estimate of net profit. Some of the balance of $25 is spent on administrative salaries not included in the salaries and wages under item *a* and office wages. A part is undoubtedly spent on financing and a part on taxes. The Bureau of Internal Revenue (Statistics of Income for 1928, Table 15) gives interest paid by construction companies as 1.04 per cent of gross, and taxes other than income tax as 0.48 per cent. Adding the income tax would bring this figure to slightly more than 1 per cent. There are, of course, a number of other minor items, but for the most part these have been included under the items previously discussed. In view of these facts, substantial accuracy would appear to be preserved if, on the undistributed balance given above ($25), $15 is assigned to office salaries and wages not included in item *a*, $5 to taxes, and $5 to interest payment. The following distribution results:

Profit as defined above (job profit)	$35.00
Office salaries and wages	15.00
Interest	5.00
Taxes	5.00
Profit (net)	10.00
	35.00

The distributions made and described above are summarized in Table 3.

Table 3.—*Summary of the various steps through which the contractor's payment of $1,000 are traced, and the amounts attributable to each*

Item	Salaries and wages	Freight	Materials and supplies	Fuel	Interest	Taxes	Depreciation and repairs	Depletion	Profit	For redistribution
a. Salaries and wages	$141.00									
b. Aggregate	50.00	$194.00	$18.00	$11.00	$4.00	$ 6.00	$29.50	$7.50	$4.00	
c. Cement	61.00	113.50	43.00	22.50	4.50	5.00	40.50	3.00	31.00	
d. Steel	6.00	6.20	8.50	1.50	.40	.90	2.50		1.00	
e. Equipment	5.20	4.50	10.65	.50	.20	7.20	70.65		1.10	
f. Plant installation	13.50	13.50								
g. Bonds and insurance	11.00								1.00	$10.00
h. Job margin	15.00				5.00	5.00			10.00	
	302.70	331.70	80.15	35.50	14.10	24.10	143.15	10.50	48.10	10.00
Adjustments[1]		+75.00	−50.00				−25.00			
	302.70	406.70	30.15	35.50	14.10	24.10	118.15	10.50	48.10	10.00

[1] These adjustments are made here to avoid recasting the freight analysis on account of freight allowances on repairs, replacements, materials, and supplies.

FREIGHT COSTS ANALYZED

For the year 1929 the Bureau of Railway Economics gives the following interesting statement of the cost elements involved in operating railroads:

[Repeated here to 2 decimal places rather than to 4 as published]

	Per cent	Distribution of $406.70 charges
Total salaries and wages, including executive management	42.42	$172.50
Locomotive fuel, principally coal	5.34	21.70
Materials and supplies	18.05	73.40
Rent of cars and common facilities	1.97	8.00
Other expense (depreciation, loss and damage, injuries)	5.66	23.00
Taxes	6.29	25.40
Rent of leased roads	2.73	11.10
Interest	7.00	28.50
Dividends	7.77	31.60
Balance	2.77	11.30
		406.70

Of the above items some, as salaries and wages, can be used in the form in which they now stand; others require some further comment. Thus, materials and supplies cover the cost of repair parts for mechanical equipment as well as the numerous materials otherwise used in connection with the operation of our railroad system.

In the absence of more accurate information about half of this item, $37, is treated as repairs and the balance, $36.40, as materials and supplies. Rent of cars and common facilities and rent of leased roads cover payments for the use of facilities not owned. Accordingly they are included with interest in the column set aside for interest and rents as these items have a somewhat similar ultimate purpose. Other expenses is made up of two quite unreliable items. Depreciation, which is an equipment charge, generates about five-sevenths of the charge; the balance is made up of charges for loss and damage, injuries and insurance. In the last analysis, loss and damage payments are purchases of materials. It is also at least measurably correct to consider payments on account of injuries as salaries and wages. Of the charge for other expense $16.50 has therefore been allocated to depreciation, $4 to materials and supplies, and $2.50 to salaries and wages. Dividends and balance given above are, of course, profit.

If the totals from Table 3 are adjusted on the basis of this distribution, the following cost distribution results:

	Amount	Redistributed railroad charges	Total
Salaries and wages	$302.70	$175.00	$477.70
Materials and supplies	30.15	40.40	70.55
Fuel	35.50	21.70	57.20
Interest, rents, etc	14.10	47.60	61.70
Taxes	24.10	25.60	49.70
Depreciation and repairs	118.15	53.50	171.65
Profit	48.10	42.90	91.00
Depletion	10.50	0	10.50
Redistribution	10.00	0	10.00
		406.70	1,000.00

FUEL COSTS REDISTRIBUTED

The latest information regarding labor and other costs incident to the production of bituminous coal is that published by the Bureau of the Census for the year 1929. Wages are reported as amounting to $574,800,072. Salaries are not reported for 1929. In 1919 they were slightly more than 10 per cent of the cost of wages. The value of the products, which includes a small amount for other than coal produced, is given as $966,693,771. It would therefore appear that salaries and wages absorb about 65 per cent of the value of the coal produced, which amounts to about 67 per cent of the coal sold. Information released by the Bureau of Internal Revenue (Statistics of Income for 1928) indicates that the industry, taken as a whole, operates at a loss. The following table gives the results of distributing the cost of fuel.

	Amount	Redistributed coal costs	Total
Salaries and wages	$477.70	$38.30	$516.00
Materials and supplies	70.55	6.65	77.20
Interest, rents, etc	61.70	2.05	63.75
Taxes	49.70	1.70	51.40
Depreciation and repairs	171.65	4.10	175.75
Profit	91.00	0	91.00
Depletion	10.50	4.40	14.90
Redistribution	10.00	0	10.00
		57.20	1,000.00

MATERIALS AND SUPPLIES, REPAIRS AND DEPRECIATION ADD TO WAGE ACCOUNT

The two remaining items that involve the direct use of commodities are materials and supplies and repairs and depreciation. The number of commodities involved in these two groups is so large that even if the amount of each kind used were known exactly, the task of dividing each one into its constituent parts would be very great. Actually, the materials used are not known in detail, though often the more important materials are a matter of rather common knowledge. On this account, it is necessary to adopt a somewhat different method for the further study of this matter, which is possible because for every commodity separately classified the Bureau of the Census, in its census of manufacturers, gives "the value added by manufacture." This value is the difference between the value of the commodities manufactured and the cost of the materials, supplies, fuel and power used in producing them. The value added by manufacture then contains, first, the salaries and wages paid; second, all charges for depreciation; third, such miscellaneous items as taxes, interest and sales expense; and finally, profit.

Taking up first the further analysis of repairs and depreciation, a number of facts are apparent. The first of these is that while charges for depreciation may accumulate as cash reserve, as a general rule they are expended on renewals and improvements about as fast as they accrue. On this account they may reasonably be considered as expenditures for new equipment. Obviously, repair costs are such expenditures.

The second point deserving comment is that, as charges for repairs and depreciation are very largely generated by the plant and equipment used in manufacturing and in transporting, so much the larger part of these charges involves such items as machine tools, foundry and machine shop products, motor vehicles, motor vehicle bodies and parts, cars (railroad) and locomotives, it will be assumed that all of this charge involves these fields.

The third point to be noted is that, in the very nature of the case, charges for repair parts and for new machines include delivery costs, freight, or express. These charges should be estimated.

Upon adjustment of the table in the light of the results of the analysis of depreciation and repairs, the following distribution of expenditures results:

	Amount	Repairs and depreciation	Total
Salaries and wages	$516.00	$52.70	$568.70
Materials and supplies	77.20	99.25	176.45
Interest, rents, etc	63.75	1.75	65.50
Taxes	51.40	4.40	55.80
Depletion	14.90	0	14.90
Profit	91.00	17.65	108.65
Redistribution	10.00	0	10.00
	....	175.75	1,000.00

A similar analysis of materials and supplies results in the adjustments shown in the following table:

	Amount	Materials and supplies	Total
Salaries and wages	$568.70	$162.20	$730.90
Interest, rents, etc	65.50	8.45	73.95
Taxes [1]	30.80	8.65	39.45
Depletion	14.90	3.05	17.95
Profit	108.65	19.10	127.75
Redistribution	10.00	0	10.00
	....	201.45	1,000.00

[1] $25 deducted on account of materials and supplies and added to this account for redistribution here.

TAXES SIMILARLY REDISTRIBUTED

Taxes are payments in support of government. Municipal, county, State, and Federal taxes are involved in unknown proportions. Interest payments absorb less than 5 per cent of the gross State revenues. They absorb something more than 15 per cent of the revenue of the Federal Government. (See Financial Statistics of States, 1929, Bureau of the Census.) The proportion of municipal and county revenue paid out as interest is unknown but is probably higher than the percentage reported as paid out by the States. In distributing taxes, it has been assumed that about 10 per cent is distributed as interest payments.

The pay roll absorbs a very large part of all government expenditures. The amount so absorbed is unknown but, aside from expenditures for public work salaries and wages are believed usually to absorb the greater part of the balance after debt service. As only a comparatively small amount of the taxes on business operations is spent in the highway field and as most other public works, except buildings and sewer systems, are presumed to be self-supporting, about 50 per cent of the tax taken out is allocated to salaries and wages and the balance—about 40 per cent—is allocated to the account of materials and supplies. On the basis of a preliminary calculation, as shown above, this was charged to materials and supplies as $25, which left a balance of $39.45 to be distributed, one-sixth to interest and five-sixths to salaries and wages. Distributed on this basis, interest amounts to $6.55 and salaries and wages to $32.90. The table now reduces to the following form:

	Amount	Distribution of taxes	Total
Salaries and wages	$730.90	$32.90	$763.80
Interest, rents, etc	73.95	6.55	80.50
Depletion	17.95	0	17.95
Profit	127.75	0	127.75
Redistribution	10.00	0	10.00
	....	39.45	1,000.00

If the $10 held for redistribution is now distributed under the headings now remaining, the following table results:

	Amount	Distribution of $10	Total
Salaries and wages	$763.80	$7.70	$771.50
Interest, rents, etc	80.50	0.80	81.30
Depletion	17.95	0.20	18.15
Profit	127.75	1.30	129.05
	....	10.00	1,000.00

In this form the original expenditure is now divided into two classes of expenditures:

Payments for salaries and wages	$771.50
Payments to owners—that is, owners of the business, of property, or of money used in the various businesses conducted	228.50
	1,000.00

This is about as far as the quantitative analysis may be carried with approximate certainty. If a further breakdown is attempted the quantities become somewhat doubtful; but there is still a further share for labor in the last quarter of the expenditure.

The preceding quantitative discussion is based on a period of unusual business activity. In times of depression such as the present, the residue composed of interest, rents, royalties, and profits shrinks both in absolute amount and in relation to the total. In view of this well known fact, it seems probable that, of the total expenditures for road construction at the present time, nearer 85 than 75 per cent may be thus directly traced into the hands of labor. Beyond this, there is still to be considered the fact that a part of the money paid to owners is immediately reinvested or expended, even in periods of depression, although a greater part is certainly so used in more prosperous times. And

(Continued on p. 38)

THE RESISTANCE OF CONCRETE TO FROST ACTION

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THIS report gives the results of an investigation which was begun about six years ago for the purpose of determining how the resistance of concrete to frost action is affected by the character of the coarse aggregate used in the mixture. The tests were the first of a number of investigations to be started, all along the same general line, and including, among others, researches by C. H. Scholer at Kansas State Agricultural College; C. A. Hughes at the University of Minnesota, and the Portland Cement Association. The work was greatly hampered at the beginning because of a lack of information as to the proper procedure to be followed in making freezing tests in the laboratory. For this reason the original freezing cycle, which consisted of alternately saturating a large volume of concrete in water at 70° F. and then subjecting the specimens to an air temperature of about 20° F. in an ordinary cold storage room, proved entirely inadequate for the purpose.

This effort was followed by attempts to produce action through the use of lower freezing temperatures; that is, a minimum of about 5° F. Even under these conditions the action proved to be so slow that the method was finally abandoned in favor of an entirely different type of freezing cycle, as well as a different type of specimen. The results of tests by the third method, while somewhat contradictory and by no means conclusive, proved of considerable interest and it has accordingly been decided to report the data obtained, together with certain results obtained during work under the second cycle which appear to be significant.

In reviewing the data presented in this report the reader should bear in mind that these tests were of an exploratory nature. Very little was known about the subject when the work was started. Many mistakes were made and many things were done which it is now realized should not have been done. It is felt, however, that the results which have been finally obtained are of sufficient significance to justify the time and expense of carrying out this series of tests. In studying the data, however, it should be borne in mind that the results of tests involving individual materials should not be interpreted too literally, but should only be considered as indicating the possibility of certain trends which are of general significance only as they are substantiated by further work on aggregates having similar characteristics.

CONCRETE SPECIMENS

The program called for the fabrication of 648 concrete beams 6 by 6 by 30 inches in size, using 18 different coarse aggregates of widely varying characteristics and three proportions, 1:1½:3, 1:2:4, and 1:3:6. Because of the fact that the stock of cement reserved for these tests was not quite sufficient to complete the series, it was found necessary to use a different cement in making up the specimens in which aggregates 2 and 15 were used. For this reason the results of freezing tests on these concretes are not included in Tables 5 and 6, although reference to the behavior of aggregate 15 in concrete is made in the discussion, because of certain interesting features in connection with this particular material. A standard Portland cement having the physical properties indicated in Table 1 and Potomac River concrete sand with properties as indicated in Table 2 were used in all specimens with the exception just noted. The coarse aggregates were all sampled at the point of origin by a representative of the bureau and were chosen so as to represent materials having a wide range in characteristics. Several sources were included which had previously been questioned because of failure in the sodium sulphate soundness test as well as others which were of unquestioned soundness. The types, character, and essential physical properties of the coarse aggregates used in these tests are given in Table 3.

TABLE 1.—*Tests of Portland cement. Results are the average of tests on six samples*

Fineness: Retained on No. 200 sieve	14.8 per cent.
Initial set	3 hours, 40 minutes.
Final set	6 hours, 25 minutes.
Tensile strength of 1.3 Ottawa sand mortar briquets:	
7 days	260 pounds per square inch.
28 days	355 pounds per square inch.

TABLE 2.—*Tests of fine aggregate. Results are the average of tests on five samples*

Sieve analysis:	Per cent
Retained on No. 10 sieve	16
Retained on No. 20 sieve	31
Retained on No. 30 sieve	52
Retained on No. 50 sieve	81
Retained on No. 100 sieve	94
Loss by elutriation	2.3
Tensile strength ratio [1] (1:3 mortar briquets):	
7 days	112 per cent
28 days	110 per cent

TABLE 3.—*Characteristics of coarse aggregates used in freezing and thawing tests*

Aggregate No.	Type	State	Mineral composition	Percentage of absorption	Percentage of wear[1]	Percentage of soft pieces[2]
1	Limestone	Maryland	Siliceous limestone	0.10	4.2	
2	Gravel	District of Columbia.	Quartz, chert, gneiss, schist, sandstone.	.51	8.9	24
3	do	South Carolina	Granular quartz	.58	32.6	91
4	do	Pennsylvania	Sandstone	1.21	12.3	20
5	do	New Jersey	Granular quartz	.51	27.6	80
6	do	Massachusetts	Quartz, granite, quartzite, schist, gneiss.	.45	18.3	28
7	Dolomite	Ohio	Dolomite	1.36	9.0	
8	Diabase	Connecticut	Diabase	.20	2.8	
9	Basalt	New Jersey	Basalt	.10	[3] 2.0	
10	Limestone	Ohio	Crystalline limestone	[4] 2.65	[4] 7.0	
11	Dolomite	Illinois	Dolomite	1.35	[3] 5.3	
12	Limestone	Minnesota	Argillaceous limestone	1.35		
13	Gravel	do	Dolomite, limestone, chert, granite, and shale.	.49	7.0	5
14	Dolomite	Michigan	Dolomite	[4] 2.21	7.6	
15	Limestone	Illinois	Chert and limestone	1.90		
16	do	New York	Cherty limestone	.08	4.2	
17	Slag	Ohio	Porous blast furnace slag.	2.85	(5)	
18	do	do	Blast furnace slag		(6)	

[1] A. A. S. H. O. tentative standard method No. 3 for stone, No. 4 for gravel.
[2] A. A. S. H. O. tentative standard method No. 6.
[3] Average of several tests from this source. Test not made on sample used in this series.
[4] Material variable in quality. These are maximum values.
(5) No abrasion test made. Weight per cubic foot approximately 70 pounds.
(6) No abrasion test made. Weight per cubic foot approximately 85 pounds.

Each of the samples of coarse aggregate was regraded at the laboratory by passing it over a revolving screen equipped with sections having ¼-inch, ½-inch, ¾-inch, and 1¼-inch square openings. The concrete aggregate in each case was made up of the three fractions resulting from this screening, in the following proportions:

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[1] As compared with 1:3 Ottawa sand mortar briquets.

	Per cent
¼-inch to ½-inch	20
½-inch to ¾-inch	25
¾-inch to 1¼-inch	55

A determination of the weight per cubic foot was made on each sample graded as above, and from this value the necessary weights of each size to be used in proportioning the concrete was calculated. Concrete was mixed with shovels in large galvanized iron pans, and molded into beam specimens 6 by 6 by 30 inches in size in accordance with a uniform procedure. Consistency was maintained by means of the flow test at approximately 140; that is, a medium consistency such as would be used in pavement work. All specimens were stored under wet earth for 28 days, after which the first freezing cycle was started. The wet earth storage was continued in the case of all specimens not being subjected to freezing and thawing.

For each aggregate 36 concrete beams were fabricated, 12 of each of the three proportions, 1:1½:3, 1:2:4 and 1:3:6. Eighteen specimens of each lot (6 of each of the 3 mixes) were subjected to freezing and 18 were stored under normal temperature conditions.

SOUNDNESS TESTS OF AGGREGATES

In addition to the various routine tests, the results of which are given in Table 3, each coarse aggregate was subjected to five reversals in the sodium sulphate soundness test conducted in accordance with the tentative standard method of the American Association of State Highway Officials[1] as well as a direct freezing test conducted in accordance with the following method.

Representative samples of each aggregate weighing approximately 10 pounds were alternately saturated in water at 70° F. and then frozen in air at a temperature of about 0° F. An ice machine having a total capacity of 4 cubic feet was used for this purpose, as well as for freezing concrete during the third cycle. The aggregates were contained in small wooden boxes having copper screen bottoms. The samples were divided into two lots, one of which was frozen 8 hours and thawed 16 hours, while the other was frozen 16 hours and thawed 8 hours. At the end of each week the lots were reversed so as to equalize the severity of the freezing action. Progressive disintegration was noted by making visual examinations from time to time of the individual fragments composing each sample The results of the sodium sulphate and freezing tests on the aggregates are given in Table 4.

FREEZING TESTS ON CONCRETE

First cycle.—As previously noted, the first freezing cycle consisted of saturating the full-size beams with water and then immediately removing them to a room having an air temperature of approximately 20° F. An attempt was made to freeze nine specimens of each lot (three of each mix) at one time, making a total of 162 specimens in the cold room, while a corresponding number were being thawed. The freezing room chosen for the work was one of the rooms in the cold storage plant of the Department of Agriculture at Arlington, Va. The room was 8 feet by 14 feet in size by 11 feet in height. Although previous tests had indicated that thorough freezing could be accomplished in about 24 hours, the capacity of the plant proved entirely inadequate for the volume of concrete which was to be frozen. This was demonstrated at the end of 50 and again at the end of 100 cycles of freezing and thawing, when tests for modulus of rupture of the frozen beams were made and the results compared to similar tests on unfrozen specimens. In neither case was there evidence of any deleterious effect of frost action, even in the 1:3:6 mixes.

TABLE 4.—*Results of sodium sulphate soundness and freezing tests on coarse aggregates*

Coarse aggregate No.	Type of coarse aggregate	Percentage absorption	Sodium sulphate soundness test	Freezing test						
				Alternations at first indication of failure	Per cent affected at end of alternations					
					21	30	40	80	230	280
1	Limestone	0.10	Sound	21[1]	5	5	5	5	5	5
2	Gravel	.51	do	21			30			30
3	do	.58	do	10			95	95	95	98
4	do	1.21	Unsound	21					50	50
5	do	.51	Sound	10			95	95	95	98
6	do	.45	do	21				40	40	40
7	Dolomite	1.36	do	(1)						
8	Diabase	.20	do	(1)						
9	Basalt	.10	do	(1)						
10	Limestone	(2)	Unsound	21				30	60	60
11	Dolomite	1.35	do	21			5	5	5	5
12	Limestone	1.35	do	21			15	100	100	100
13	Gravel	.49	do	10	5	5	5	5	5	5
14	Dolomite	2.24	do	10		30	30	80	80	80
15	Limestone	1.90	do	10			80			85
16	do	.08	Sound	80				5	5	5
17	Slag	2.85	do	21				10	10	30
18	do	(3)	(3)	21	1	1	1	1	1	60

(1) No action. (2) Variable; see discussion. (3) Test not made.

Second cycle.—It was realized after these experiments that in order to increase the efficiency of the freezing plant to a point where it would be possible actually to freeze the specimens, it would be necessary to reduce very materially the number frozen at one time. In view of the time required to do this, it was decided to try freezing a full set of the 1:3:6 specimens in the cold-storage plant at the old Center Market, Washington, D. C., where it was possible to obtain not only lower temperatures (a minimum of about 5° F.) but also much greater capacity, resulting in more rapid freezing. This cycle consisted of alternately immersing in water at 70° F. and freezing in air at a minimum of 5° F. The duration of each complete alternation was 24 hours. The specimens subjected to this test were of the 1:3:6 mix and comprised beams which had already been subjected to 100 alternations at Arlington. Three beams for each aggregate were subjected to this cycle and at the conclusion of 70 alternations were tested for modulus of rupture in comparison with similar beams unfrozen. The concrete specimens at this time were approximately two years old. The results of these tests are shown in Table 5. It will be observed that there is, in all cases, a falling off in strength due to freezing. The extent of this reduction varies from 3 per cent in the case of aggregate No. 10, to 100 per cent in the case of aggregate No. 14. However, an examination of the data fails to show any definite relation between the soundness of the aggregate as revealed by either the sodium sulphate test or a freezing test and the soundness of the concrete. An inspection of the frozen specimens after test indicated that the reduction in strength was, in almost all cases, due to a weakening of the mortar rather than to the coarse aggregate. Exception to this conclusion may be noted in aggregate No. 14, a silicious and argillaceous dolomite. In this case action had progressed to the point where the concrete failed before any load could be applied. Failure was undoubtedly accelerated by action on the aggregate. It is of interest to note that concrete containing the two limestones and the dolomite which were sound by the sodium sulphate test (Nos. 1, 7, and 16) showed relatively high resistance to frost action, as did also one unsound limestone, No. 10. The two trap rock concretes, Nos. 8 and 9, although the coarse aggregates were of unquestioned soundness, showed low resistance, possibly because of some condition associated with bond between mortar and aggregate.

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[1] A. A. S. H. O. Tentative Standard method of test for soundness of coarse aggregate (T-9).

TABLE 5.—*Effect of alternate freezing and thawing on the flexural strength of concrete containing various types and grades of coarse aggregate. Each value average of tests on three specimens.*

Proportions	1:3:6 by volume.
Consistency	Medium.
Water-cement ratio	Approximately 1.40.
Age at test	Approximately 2 years.
Freezing treatment	100 alternations, cycle No. 1 70 alternations, cycle No. 2

Coarse aggregate No.	Type of coarse aggregate	Rating of coarse aggregate, sodium sulphate test	Modulus of rupture in pounds per square inch		Percentage reduction in strength
			Frozen	Unfrozen	
1	Limestone	Sound	354	373	5
3	Gravel	do	264	340	22
4	do	Unsound	266	348	24
5	do	Sound	271	340	20
6	do	do	244	384	14
7	Dolomite	do	390	435	10
8	Diabase	do	272	383	29
9	Basalt	do	341	418	18
10	Limestone	Unsound	435	440	3
11	Dolomite	do	341	452	25
12	Limestone	do	272	384	29
13	Gravel	do	360	478	25
14	Dolomite	do	(1)	454	
16	Limestone	Sound	362	432	16
17	Slag	do	342	435	21
18	do		337	448	25

[1] Concrete failed before load could be applied.

Third cycle.—The test results which are reported in Table 5 indicated that, even under the low-temperature condition obtaining at the Center Market, action would be very slow on the richer mixtures. It was accordingly decided to abandon this method of freezing entirely in favor of some method which would insure a more accelerated action. For this purpose the small ice machine used for the freezing tests on the aggregates was employed. The specimens for freezing were prepared by sawing concrete beams which had been stored under earth continuously until this time into prisms, 4 inches in width. This provided specimens 6 by 6 by 4 inches in size, each of which had two sawed faces, exposing the coarse aggregate, three faces which had been in contact with the form and one face which was the surface of the original beam as molded. The specimens were sawed with a large circular saw set with carborundum teeth. Four specimens for each aggregate, two of each of the 1:2:4 and 1:1½:3 mixes, were prepared for test. The group of thirty-six 1:2:4 specimens was tested first, the alternations being so arranged that one specimen of each aggregate was freezing while the other was thawing. This provided for the freezing of 18 specimens at one time. The freezing of the 1:1½:3 specimens followed the completion of 200 alternations on the 1:2:4 concretes. The 1:1½:3 specimens were frozen for a total of 130 alternations. Figure 1 illustrates the appearance of the prisms before test. The varied effects of alternate freezing and thawing are shown in Figure 2.

TABLE 6.—*Relation between soundness of coarse aggregates and resistance of concrete to freezing and thawing. Age at test, approximately 3 years*

Aggregate No.	Soundness tests on aggregate			Freezing and thawing tests on concrete					
				[1] Mix, 1:1½:3			[1] Mix, 1:2:4		
	Type	Rating by sodium sulphate test (5 cycles)	Rating by freezing test [3]	Number of alternations at failure[2]		Coarse aggregate failed	Number of alternations at failure		Coarse aggregate failed
				Initial	Final		Initial	Final	
						P. ct.			*P. ct.*
1	Limestone	Sound	Sound (5)	40	100	0	55	90	0
3	Gravel	do	Unsound (98)	50	90	0	40	65	0
4	do	Unsound	Unsound (50)	70	110	1	60	110	5
5	do	Sound	Unsound (98)	70	110	0	50	100	0
6	do	do	Questionable (40)	40	80	0	30	70	0
7	Dolomite	do	Sound (0)	40	90	0	30	90	0
8	Diabase	do	do	40	90	0	20	65	0
9	Basalt	do	do	40	100	0	20	65	0
10	Limestone	Unsound	Unsound (60)	85	(4)	5	50	90	5
11	Dolomite	do	Sound (5)	85	(5)	10	50	150	10
12	Limestone	do	Unsound (100)	40	100	100	20	70	100
13	Gravel	do	Sound (5)	40	(6)		35	200	(7)
14	Dolomite	do	Unsound (80)	85	(8)	50	85	160	20
16	Limestone	Sound	Sound (5)	85	(9)	10	55	160	10
17	Slag	do	Sound (30)	40	130		22	65	
18	do		Questionable (60)	15	80		35	65	

[1] The average water-cement ratio (uncorrected for absorption) was approximately 0.8 for the 1:1½:3 concretes and approximately 1 for the 1:2:4 concretes.

[2] Freezing tests on the 1:1½:3 specimens were discontinued at the expiration of 130 alternations.

[3] Figures in parentheses indicate percentage of total sample affected at the end of 280 alternations. (See Table 4.)

[4] 60 per cent disintegrated at 130 alternations.

[5] 90 per cent disintegrated at 130 alternations.

[6] Bottom of specimen frozen in water for 10 alternations. Failure at 110.

[7] No quantitative determination. A number of fragments softened.

[8] 40 per cent disintegrated at 130 alternations.

[9] 30 per cent disintegrated at 130 alternations.

One complete alternation was accomplished in 24 hours. The time of freezing for each lot was alternated from week to week between 8 hours and 16 hours; that is, during the first week one lot would freeze 8 hours and thaw 16 hours, and the other lot freeze 16 hours and thaw 8 hours. During the second week the lots were reversed so that eventually each specimen averaged 12 hours freezing and 12 hours thawing in each 24-hour cycle.

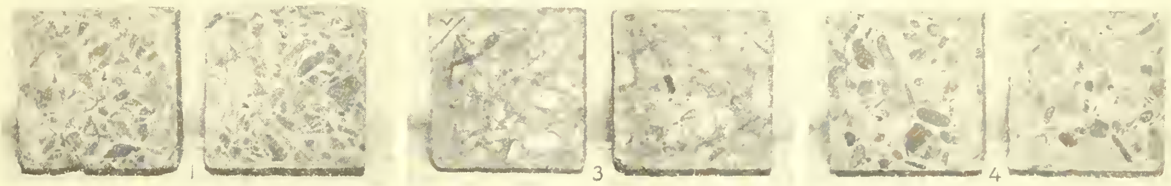
The position of the prisms in the freezing cans was changed daily. The prisms were placed in the cans in three tiers, three prisms in each tier, arranged so as to have a 1-inch air space surrounding all specimens. Tests for rate of freezing made by means of thermocouples installed at the center of the prism indicated that the temperature of the concrete reached the freezing point approximately one hour after being placed, in the cans. From this point the temperature dropped at the rate of about 9° F. per hour to a minimum of about −4°F., which was maintained until the end of the cycle.

There follows a brief summary of observations taken during the freezing test on the prisms which gives, for each coarse aggregate, an indication of the rate at which disintegration developed, as well as a statement of the general characteristics of the coarse aggregate, its rating by the sodium sulphate soundness test, and the results of direct freezing tests on the aggregate. The data given in this summary are also given in tabular form (Tables 3, 4, and 6) except that tabular results for the concrete freezing tests shown in Table 6 give only the number of alternations at first indication of failure and at final failure. Absorption and abrasion tests on the coarse aggregate are shown in Table 3 and the results of sodium sulphate and freezing and thawing tests on the aggregate in Table 4.

As stated before, there were two prisms for each aggregate and each of the two mixes, 1:2:4 and 1:1½:3. In making the studies on which the summaries which follow are based, both prisms of each group were examined visually and the data given constitute averages for the two.

DETAILED OBSERVATIONS ON SODIUM SULPHATE SOUNDNESS AND FREEZING TESTS ON AGGREGATES AND ON FREEZING TESTS OF 1:1½:3 AND 1:2:4 CONCRETE

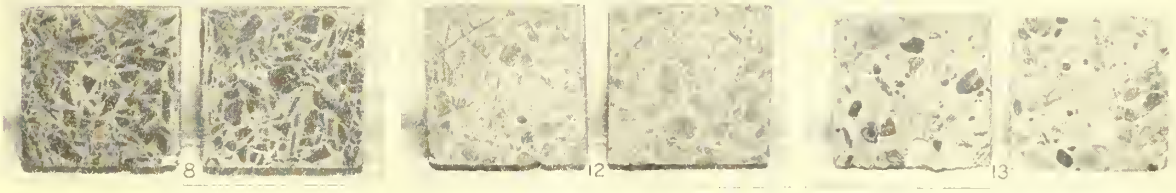
Aggregate No. 1. Siliceous limestone from Frederick, Md.—(*a*) General characteristics: A hard siliceous limestone with a percentage of wear of 4 2 and a low absorption of 0.10. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on the aggregate: At 21 alternations about 5 per cent of sample showed failure by splitting along cleavage lines on planes of weakness. No further action at the end of 280 alternations. Rated as sound by freezing test. (*d*) Freezing tests on concrete: Tops and sides of prisms began to scale at about 40 alternations in the case of the 1:1½:3 prisms and at about 55 alternations in the case of the 1:2:4 prisms. The 1:2:4 prisms completely disintegrated at the end of 90 alternations. The 1:1½:3 prisms disintegrated at 100 alternations. Failure due in both cases to expansion of mortar on freezing, causing splitting and disintegration of concrete. Coarse aggregate entirely unaffected. Disintegration of the 1:1½:3 prisms seemed to progress somewhat faster in certain stages than that of the 1:2:4 specimens, although difference in behavior was not marked.



Silicious Limestone

Gravel Containing Quartz, Chert, Gneiss, Schist, and Sandstone

Sandstone Gravel



Diabase

Argillaceous Limestone

Gravel Containing Dolomite, Limestone, Chert, Granite, and Shale

Figure 1.—Examples of Concrete Prisms Used in Freezing Tests, Showing Sawed Surface With Aggregate Exposed. Numbers Correspond With Those Given in Table 3

Aggregate No. 3. Gravel from Wateree, Richland County, S. C.—(*a*) General characteristics: A gravel consisting essentially of very soft, friable fragments of sandstone and so-called sugary quartz. Sample showed a percentage of wear of 32.6 by modified Deval test and an absorption of 0.58. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on the aggregate: Softening of individual pieces of sandstone noted at 10 alternations. Ninety-five per cent of sample affected at 40 alternations. About 2 per cent sound at the end of 280 alternations. Rated as unsound by freezing test. (*d*) Freezing tests on concrete: Top and sides of 1:2:4 prisms began to scale at about 40 alternations. Corners and edges of 1:1½:3 prisms began to spall at about 50 alternations. 1:2:4 prisms 50 per cent disintegrated in 50 alternations and completely disintegrated in 65 alternations. The 1:1½:3 prisms were somewhat more resistant, total failure occurring at 90 alternations. In neither case was the coarse aggregate affected in any way. Failure occurred through softening of mortar, followed by loosening of the bond between the mortar and the coarse aggregate.

Aggregate No. 4. Gravel from Allegheny River, near Pittsburgh, Pa.—(*a*) General characteristics: Gravel consisting of rounded fragments of sandstone with some quartz. Percentage of wear, modified Deval test, 12.3; absorption 1.21. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on the aggregate: Softening of some of the sandstone fragments noted at 21 alternations. Action progressive from that point with about 50 per cent of entire sample affected at 230 alternations. Very little further action at 280 alternations. Large pieces of sandstone weathered slightly on surface but did not soften or crack. Rated as unsound in freezing test. (*d*) Freezing tests on the concrete: First failure in 1:2:4 prisms noted at about 60 alternations and in 1:1½:3 prisms at about 70 alternations. Top of prism scaled. Corners and edges spalled. Action from this point progressed up to total failure of both sets at approximately 110 alternations. A very small percentage of the coarse aggregate (1 to 5 per cent) affected. Failure due to softening of mortar and loosening of bond between mortar and coarse aggregate.

Aggregate No. 5. Gravel from Millville, N. J.—(*a*) General characteristics: Gravel consisting essentially of rounded fragments of sugary quartz, very friable. Percentage of wear, 27.6 by modified Deval test. Absorption, 0.51 per cent. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: First failure at 10 alternations, 95 per cent total sample affected at 40 alternations, 98 per cent at 280 alternations. Rated as unsound in freezing test. (*d*) First failure on 1:2:4 prisms at about 50 alternations, on 1:1½:3 prisms at about 70 alternations. Corners spalling; tops of 1:2:4 prisms scaling. 1:2:4 prisms about 25 per cent disintegrated at 90 alternations and completely disintegrated at 100 alternations. 1:1½:3 prisms about 10 per cent disintegrated at 80 alternations and 100 per cent disintegrated at 110 alternations. Action somewhat slower on richer concrete. Mortar failure in both cases. No action on coarse aggregate.

Aggregate No. 6. Gravel from North Wilbraham, Mass.—(*a*) General characteristics: Glacial gravel consisting essentially of rounded fragments of quartz, quartzite, schist, and gneiss. Percentage of wear 18.3 by modified Deval test. Absorption 0.45 per cent. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: First evidence of failure at 21 alternations, slight softening of surface of some fragments. At 80 alternations about 40 per cent of entire sample affected. No further action at 280 alternations. Rated as of questionable soundness by freezing test. (*d*) Freezing test on concrete: Top of 1:2:4 prisms scaling at corners and spalling at 30 alternations. Corners and edges of 1:1½:3 prisms spalling at 40 alternations. Progressive failure on both sets resulting in total failure of 1:2:4 prisms at 70 alternations and total failure of 1:1½:3 prisms at 80 alternations. No action on coarse aggregate, failure being due to disintegration of mortar.

Aggregate No. 7. Dolomite from Forest, Ohio.—(*a*) General characteristics: Dolomite, uniform in color, hardness, and composition. Percentage of wear, 9; absorption, 1.36. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: No action to 280 alternations. (*d*) Freezing test on concrete: Top of 1:2:4 prisms scaling with corners and sides spalling at 30 alternations. Similar action on 1:1½:3 prisms at 40 alternations, except that top did not scale. Both sets completely disintegrated at 90 alternations. No action on coarse aggregate. Mortar softened between particles of coarse aggregate. Bond between mortar and coarse aggregate, however, did not fail, as mortar continued to adhere to surface of coarse aggregate even after failure of prisms.

Aggregate No. 8. Diabase (trap) from New Haven, Conn.—(*a*) General characteristics: Diabase, uniform in color, hardness, and composition. Percentage of wear, 2.8; absorption, 0.20. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: No action in 280 alternations. (*d*) Freezing test on concrete: First signs of failure in 1:2:4 prisms at 20 alternations, followed by rapid progressive disintegration. Ninety per cent disintegrated at 50 alternations and completely disintegrated at 65 alternations. Action on 1:1½:3 prisms started at 40 alternations with 90 per cent disintegrated at 80 alternations followed by complete failure at 90 alternations. No action on coarse aggregate. Bond between mortar and coarse aggregate loosened.

Aggregate No. 9. Basalt (trap) from Bound Brook, N. J.—(*a*) General characteristics: Basalt of uniform color, hardness, and composition. Percentage of wear, 2; absorption, 0.10 per cent. (*b*) Sodium suphate test: Sound. (*c*) Freezing test on aggregate: No action in 280 alternations. (*d*) Freezing test on concrete: Initial failure of 1:2:4 prisms at 20 alternations, followed by progressive action, with 90 per cent disintegration at 55 alternations and 100 per cent disintegration at 65 alternations. The 1:1½:3 prisms showed initial failure at 40 alternations, with 90 per cent disintegration at 85 alternations and 100 per cent disintegration at 100 alternations. Mortar softened and loosened from coarse aggregate, which was unaffected. Behavior almost identical with that observed in case of aggregate No. 8.

Aggregate No. 10. Limestone from Bucyrus, Ohio.—(*a*) General characteristics: Limestone varying considerably in quality, ranging from argillaceous with percentage of wear equaling 3.6 and absorption of 0.59 to white crystalline with percentage of wear equaling 7 and absorption 2.65 per cent. Argillaceous material approximately 50 per cent of total. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on aggregate: Several particles disintegrated at 21 alternations; 30 per cent softened at 80 alternations and 60 per cent at 280 alternations. The argillaceous material completely disintegrated, with white crystalline fraction weakened. Rated as unsound in freezing test. (*d*) Freezing test on concrete: First failure of 1:2:4 prism at 50 alternations; sides and top scaling. This was followed by rapid progressive failure so that 90 per cent was disintegrated at 70 alternations and 100 per cent at 90 alternations. The action on the 1:1½:3 prism was somewhat slower, initial scaling and spalling not being noted until after 85 alternations. Sixty per cent of prisms had disintegrated at 130 alternations. About 5 per cent of coarse aggregate (argillaceous) affected. Failure due to softening of mortar. Mortar still adhered to particles of coarse aggregate, indicating good bond.

Aggregate No. 11. Dolomite from Thornton, Ill.—(*a*) General characteristics: Dolomite. Percentage of wear 5.3; Absorption, 1.35. About 50 per cent of total sample very porous. Some carbonaceous matter present. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on aggregate: Several pieces containing carbonaceous matter softened at end of 21 alternations. No further action at 280 alternations. Rated as sound in freezing test. (*d*) Freezing test on concrete: Initial failure of 1:2:4 prism at about 50 alternations consisting of scaling and surface pitting. Prisms disintegrated 50 per cent at 140 alternations, and 100 per cent at 150 alternations. Mortar failure with good bond, however, between mortar and coarse aggregate. About 10 per cent of aggregate affected. The 1:1½:3 prisms showed first signs of failure (scaling and pitting) at 85 alternations, with 90 per cent prism disintegrated at 130 alternations. Behavior similar to 1:2:4 prisms except that initial action was somewhat delayed.

Aggregate No. 12. Argillaceous Limestone from St. Paul, Minn.—(*a*) General characteristics: A dark argillaceous limestone, uniform in color, hardness, and composition. Absorption 1.35 per cent. No value for percentage of wear reported. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on aggregate: Indications of surface weathering of individual pieces at 21 alternations; 15 per cent of sample disintegrated at 40 alternations and 100 per cent disintegrated at 80 alternations. Rated as unsound in freezing test. (*d*) Freezing test on concrete: Surface pitting on 1:2:4 prism with softening of exposed coarse aggregate noted at 20 alternations. Prisms 70 per cent disintegrated at 55 alternations and 100 per cent disintegrated at 70 alternations. All coarse aggregate softened to putty-like consistency. Similar action on 1:1½:3 prisms, except that initial action was considerably delayed. Initial failure noted at 40 alternations; prisms 40 per cent disintegrated at 85 alternations and completely disintegrated at 100 alternations. Failure by action on both mortar and coarse aggregate.

Aggregate No. 13. Gravel from Appleton, Minnesota.—(*a*) General characteristics: Rounded fragments of dolomite, limestone, chert, granite and shale. Percentage of wear, modified Deval test, 7; absorption 0.49 per cent. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on aggregate: Pieces of shale (about 1 per cent of total) disintegrated at 10 alternations. About 5 per cent (schists and sandstone) disintegrated at 21 alternations. No further action to 280 alternations. Rated as sound in freezing test. (*d*) Freezing test on concrete: Initial failure due to popping out of exposed shale fragments from surface. Observed on 1:2:4 prism at 35 alternations; on 1:1½:3 prism at 40 alternations. 1:2:4 prisms 5 per cent disintegrated at 140 alternations and completely disintegrated at 200 alternations. Observations on 1:1½:3 prisms after 40 alternations meaningless because of error in freezing.

Aggregate No. 14. Dolomite from Monroe, Mich.—(*a*) General characteristics: Siliceous and argillaceous dolomite with cavities. Thin bedding planes of argillaceous material. Percentage of wear, 7.6; absorption, 2.24. (*b*) Sodium sulphate test: Unsound. (*c*) Freezing test on aggregate: First failure, consisting of softening of argillaceous material, noted at 10 alternations. At 30 alternations 30 per cent of entire sample affected, with 80 per cent affected at 80 alternations. Siliceous dolomite unaffected. No further action to 280 alternations. Rated as unsound by freezing test. (*d*) Freezing test on concrete: Initial failure (scaling and spalling) on both sets of prisms at about 85 alternations. Forty per cent of the 1:2:4 prisms disintegrated at 140 alternations and 25 per cent of the 1:1½:3 prisms disintegrated at 120 alternations. The 1:2:4 prisms completely disintegrated at 160 alternations. Forty per cent of the 1:1½:3 prisms disintegrated at 130 alternations. Unsound particles of coarse aggregate in both prisms disintegrated. From 20 to 50 per cent of the siliceous dolomite was affected.

Aggregate No. 16. Limestone from Buffalo, N. Y.—(*a*) General characteristics: Dark gray limestone containing flint. Percentage of wear, 4.2; absorption, 0.08 per cent. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: First indication of failure at 80 alternations. About 5 per cent of material spalled along cleavage lines. No further action at 200 alternations. At 280 alternations 50 per cent of sample showed a softening along edges of individual pieces. Rated as sound in freezing test. (*d*) Freezing test on concrete: The 1:2:4 prisms showed scaling on top at about 55 alternations; 10 per cent disintegrated at 90 alternations, with two pieces of exposed chert cracked. Prisms 50 per cent disintegrated at 140 alternations and 100 per cent disintegrated at 160 alternations. Mortar softened. Bond with chert particles broken. Number of chert fragments cracked and split. The 1:1½:3 prisms showed first signs of failure at about 85 alternations (mortar pitting on surface). Prisms were 30 per cent disintegrated at 130 alternations, with particles of chert cracked and split.

Aggregate No. 17. Blast furnace slag, from Hubbard, Ohio.—(*a*) General characteristics: A porous blast furnace slag, weight per cubic foot about 70 pounds. Absorption 2.85 per cent. (*b*) Sodium sulphate test: Sound. (*c*) Freezing test on aggregate: First evidence of failure at 21 alternations. At 80 alternations about 10 per cent of sample (the light honeycombed pieces) weakened. At 280 alternations about 30 per cent of particles weakened. Rated as sound in freezing test. (*d*) Freezing test on concrete: The 1:2:4 prisms began scaling at 22 alternations. At 50 alternations about 40 per cent of prisms disintegrated. Prisms 100 per cent disintegrated at 65 alternations. Mortar failure. Coarse aggregate unaffected.

Failure of 1:1½:3 prisms started at 40 alternations. At 80 alternations prisms had disintegrated 10 per cent. Total disintegration at 130 alternations.

Aggregate No. 18. Blast Furnace slag from Lectonia, Ohio.—(*a*) General characteristics: Blast furnace slag. Weight per cubic foot approximately 85 pounds. (*b*) Sodium sulphate soundness test not made. (*c*) Freezing test on aggregate: First evidence of failure at 21 alternations. About 1 per cent of total sample (light, porous pieces) weakened. No noticeable further action until 280 alternations, at which examination showed about 60 per cent of sample weakened. Rated as of questionable soundness in freezing test. (*d*) Freezing test on concrete: The 1:2:4 prisms showed scaling on top and sides at 35 alternations. Prisms 50 per cent disintegrated at 55 alternations and completely disintegrated at 65 alternations. Mortar failure, although some of the lighter slag fragments were weakened. The 1:1½:3 prisms started pitting and scaling at 15 alternations. Prisms were 100 per cent disintegrated at 80 alternations. A mortar failure, although light honeycombed pieces of slag were affected.

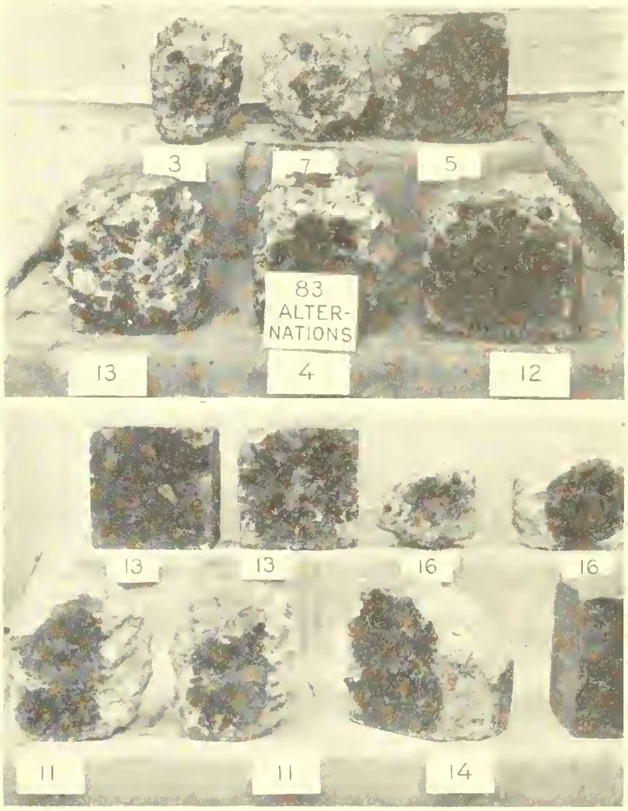


FIGURE 2.—EXAMPLES TYPICAL OF CONCRETE PRISMS AFTER ALTERNATE FREEZING AND THAWING. NUMBERS CORRESPOND WITH THOSE GIVEN IN TABLE 3. PRISMS IN UPPER PHOTOGRAPH, 1:1½:3 MIX AFTER 83 ALTERNATIONS; LOWER PHOTOGRAPH, 1:2:4 MIX AFTER 140 ALTERNATIONS

DISCUSSION

The most interesting fact to be noted from a study of these results is that, in spite of the wide variation in the quality of the coarse aggregate used, failure of the concrete due to freezing and thawing in practically all instances was due to a weakening and consequent breaking down of the mortar portion of the concrete. Although a number of the coarse aggregates were composed of either very soft or friable pieces, or showed unsoundness by the sodium sulphate test, the concretes in which these materials were used were, in general, just as resistant as concrete containing coarse aggregate of known durability. This applied to both the 1:2:4 and the 1:1½:3 mixtures, although a study of the data will show in almost all cases greater resistance in the

1:1½:3 mixes. This is due, no doubt, to the fact that in the richer mixtures the aggregates were incorporated in a paste having a considerably lower water-cement ratio, with consequently increased resistance to frost action.

Included in this series of tests were six aggregates which were rated as unsound in the sodium sulphate test. Two of these were gravels and four limestones. In only one case (aggregate No. 12), an argillaceous limestone, is there any indication that failure may have been accelerated by the character of the coarse aggregate. However, even in this case as many alternations were required to disintegrate the concrete as were necessary in the case of the concrete containing the trap rock aggregates, Nos. 8 and 9, both of which were of unquestioned soundness. Aggregate No. 14, which was also an argillaceous material, showed up very well in cycle No. 3, although the 1:3:6 specimens containing this aggregate were very badly affected as evidenced by the failure of the beams in cycle No. 2 to stand any appreciable load. (See Table 5.) This is an apparent reversal in behavior which it is rather difficult to explain. It is of course possible that the specimens had become weakened through action on the coarse aggregate some time before actual disintegration took place. It is interesting to note, however, that no evidence whatever of weathering (that is, surface scaling, pitting, etc.), which in most cases appeared at about 40 alternations, was observed until 85 alternations had been made. This would certainly indicate that the unsoundness noted in this particular aggregate was not accompanied by appreciable volume change, such as occurs when certain varieties of chert are subjected to frost action.

Gravels No. 3 and No. 5 are of interest because they represent a certain type of very friable quartz gravel known as sugary quartz which occurs along the Atlantic seaboard. Because of their extreme friability the use of these aggregates has been questioned by many engineers. These tests indicate that, although the aggregates failed in a direct freezing test, they were not of such a character as to cause unsoundness in the concrete. The same is true of aggregate No. 4, from the Allegheny River, near Pittsburgh. Although showing unsound in the sodium sulphate test, this unsoundness did not apparently contribute in any way to the failure of the 1:1½:3 or 1:2:4 concrete prisms. With the possible exception of aggregate No. 12, this seemed to be true also of all of the other aggregates which were rated as unsound in the sodium sulphate test. Here again, this particular behavior is due probably to the fact that in none of these cases did unsoundness manifest itself by violent volume change in the coarse aggregate. The results indicate that where the type of material is such that weathering produces merely a softening of the fragments, unaccompanied by volume change, the ultimate behavior of the concrete will be controlled largely by the quality of the cement matrix, rather than by the quality of the aggregate. It should be remembered that no attempt was made to include manifestly unsatisfactory materials, such as shale, in this series of tests. The idea was rather to include a number of so-called border-line materials—materials which are frequently rejected because of failure to meet some arbitrary laboratory test but concerning the real value of which we have little data.

In this connection it is of interest to call attention to the results which were secured on aggregate No. 15, a cherty limestone from Quincy, Ill. These results, as well as those for aggregate No. 2, Potomac River gravel, were not included in Tables 5 and 6 because a different cement was used in making up the concrete and direct comparisons are therefore not possible. However, observations of the concrete prisms containing aggregate No. 15 indicated beyond doubt that disintegration in this particular coarse aggregate was accompanied by volume change. In both prisms the first noticeable action was a popping out of exposed chert particles in a manner similar to that observed in actual service. This action started at five alterations of freezing and thawing. This particular aggregate contained approximately 70 per cent chert by weight.

It is realized that the method of evaluating the results of freezing tests by means of a visual examination of the specimen has many drawbacks. It involves the personal equation to a considerable extent and also fails to express quantitatively any weakening of the concrete due to frost action which is not manifested in actual disintegration. At the time these tests were made, however, the bureau's facilities for conducting freezing tests did not permit of the storing of specimens of sufficient size for strength tests after freezing.

In spite of these objections it is felt that the observations made in connection with this work were sufficiently precise to enable us to draw a reasonably accurate picture of the relative behavior of these specimens in the freezing test. The tests were not extensive enough to warrant drawing definite conclusions. It is believed, however, that certain trends have been indicated with sufficient definiteness to throw some additional light on this particular problem. These indications are as follows:

1. That, within the range in variation of aggregate quality covered by these tests, variations in the quality of mortar caused by changes in the water-cement ratio of the cement paste will have a greater effect upon the resistance of concrete to frost action than will variations in the type and character of the coarse aggregate.
2. That failure of coarse aggregates in the sodium sulphate soundness test is not necessarily an indication that the aggregate is unsatisfactory for use in concrete to be exposed to the weather.

(Continued from p. 31)

since, of the money so reinvested in productive industry, labor again receives the major part, it is not unreasonable to suggest that as much as 90 per cent and probably more of the original expenditure for a concrete pavement ultimately finds its way into wages and salaries and that this percentage is not greatly changed by the turn of the cycle from prosperity to depression and back again.

MOTOR-VEHICLE REGISTRATIONS, 1931 [1]

[Compiled for calendar or registration year from reports of State authorities]

State	1931 registered motor vehicles individually and commercially owned [2]			Other registered vehicles		Tax-exempt official motor vehicles and motor cycles		Number of licenses or permits		1930 grand total registered motor cars and trucks	Year's change in motor-vehicle registrations; increase, or decrease (−)	
	Grand total registered motor cars and trucks	Passenger automobiles, taxis, and busses [3]	Motor trucks and road tractors	Trailers	Motor cycles	United States, State, and local [4]	Motor cycles (official)	Dealers	Operators and chauffeurs		Number	Per cent
Alabama	246,465	212,493	33,972	3,279	593	1,036	----------	3,195	2,153	277,146	−30,681	−11.1
Arizona	105,572	92,939	12,633	1,623	379	1,594	53	127	34,254	110,525	−4,953	−4.5
Arkansas	180,731	149,456	31,275	3,745	365	389	----------	353	4,150	220,204	−39,473	−17.9
California	2,043,281	1,798,068	245,213	55,024	8,970	18,459	1,064	3,269	----------	2,041,356	1,925	.1
Colorado	308,458	276,376	32,082	258	962	283	----------	3,196	6,255	308,509	−51	−.02
Connecticut	336,040	284,081	51,959	926	2,097	2,032	253	2,747	416,796	331,026	5,014	1.5
Delaware	55,202	45,211	[5] 9,991	517	320	475	----------	614	68,862	56,109	−907	−1.6
Florida	323,260	271,536	51,724	6,751	1,034	3,785	131	1,668	1,818	327,801	−4,541	−1.4
Georgia	320,840	274,576	46,264	3,317	1,190	934	----------	2,324	3,544	341,580	−20,740	−6.1
Idaho	111,663	96,228	15,435	7,558	357	1,501	9	352	757	119,077	−7,414	−6.2
Illinois	1,612,770	1,411,261	201,509	9,283	5,811	979	----------	3,883	93,059	1,638,260	−25,490	−1.6
Indiana	862,672	732,846	129,826	19,169	2,781	9,066	----------	2,612	164,611	875,763	−13,091	−1.5
Iowa	748,438	670,024	78,414	3,207	1,705	4,028	76	1,834	17,056	778,386	−29,948	−3.8
Kansas	559,176	478,692	[5] 80,484	1,778	1,050	3,242	38	1,889	741,138	594,523	−35,347	−5.9
Kentucky	327,326	292,357	34,969	(6)	804	2,557	----------	841	9,535	331,002	−3,676	−1.1
Louisiana	263,050	215,267	47,783	5,670	767	209	----------	374	22,852	275,283	−12,233	−4.4
Maine	188,238	149,467	38,771	3,053	1,168	1,828	91	1,269	231,201	186,157	2,081	1.1
Maryland	325,372	289,292	36,080	1,128	1,739	1,969	----------	5,575	80,748	321,702	3,670	1.1
Massachusetts	840,190	736,302	103,888	650	3,068	1,756	----------	3,360	1,024,304	846,206	−6,016	−.7
Michigan	1,230,980	1,078,345	[5] 152,635	61,932	3,236	371	----------	1,718	----------	1,328,209	−97,229	−7.3
Minnesota	720,401	611,966	108,435	18,291	1,749	1,285	----------	2,034	47,659	732,972	−12,571	−1.7
Mississippi	183,650	152,929	[5] 30,721	2,560	180	74	----------	2,555	----------	237,094	−53,444	−22.5
Missouri	752,805	656,830	[5] 95,975	5,469	1,655	2,253	17	2,075	34,405	761,600	−8,795	−1.2
Montana	127,166	103,129	[5] 24,037	59	216	1,647	7	544	311	135,168	−8,002	−5.9
Nebraska	416,131	356,283	59,848	15,737	897	1,832	----------	2,797	----------	426,229	−10,098	−2.4
Nevada	32,168	25,218	6,950	411	75	556	----------	87	----------	29,645	2,523	8.5
New Hampshire	111,510	92,839	18,671	1,137	1,019	428	44	491	129,728	112,183	−673	−.6
New Jersey	869,867	736,506	133,361	2,916	5,738	8,708	----------	3,055	1,063,073	852,850	17,017	2.0
New Mexico	81,325	65,441	15,884	778	218	736	----------	125	----------	84,150	−2,825	−3.4
New York	2,297,249	1,966,436	330,813	12,596	12,521	22,357	1,185	4,996	3,067,853	2,307,730	−10,481	−.5
North Carolina	428,737	374,162	54,575	8,118	1,245	8,429	----------	4,348	----------	453,241	−24,504	−5.4
North Dakota	171,293	144,705	26,588	(7)	229	3	----------	427	----------	183,019	−11,726	−6.4
Ohio	1,710,625	1,518,696	[5] 191,929	32,717	6,395	14,666	----------	3,416	2,279	1,759,363	−48,738	−2.8
Oklahoma	482,725	428,140	54,585	(7)	1,200	530	----------	----------	----------	550,331	−67.606	−12.3
Oregon [8]	278,225	255,275	22,950	2,167	1,691	2,768	----------	508	52,637	[9] 273,625	4,600	1.7
Pennsylvania	1,741,942	1,522,130	219,812	6,308	11,778	2,640	393	10,255	2,214,008	1,753,521	−11,579	−.7
Rhode Island	137,878	118,313	19,565	64	775	979	112	311	164,784	136,423	1,455	1.1
South Carolina	203,719	180,280	23,439	2,100	492	3,515	84	537	39,139	218,402	−14,683	−6.7
South Dakota	193,025	169,509	23,516	4,994	283	1,098	11	710	----------	205,172	−12,147	−5.9
Tennessee	350,520	316,544	33,976	2,696	1,285	5,852	100	545	----------	368,259	−17,739	−4.8
Texas	1,297,301	1,086,310	210,991	33,861	3,727	2,505	----------	3,252	43,659	1,365,896	−68,595	−5.0
Utah	108,958	91,381	17,577	767	478	1,373	----------	300	----------	113,997	−5,039	−4.4
Vermont	83,877	75,424	8,453	457	502	28	----------	318	98,184	86,624	−2,747	−3.2
Virginia	379,227	322,594	56,633	1,582	2,134	4,868	154	3,828	14,550	375,889	3,338	.9
Washington	420,878	360,796	60,082	3,000	1,696	6,764	1,696	3,383	----------	446,062	−25,184	−5.6
West Virginia	253,308	213,949	39,359	1,270	1,190	2,619	113	8,507	48,358	266,273	−12,965	−4.9
Wisconsin	754,249	640,476	113,773	1,007	2,316	5,657	289	2,678	----------	782,562	−28,313	−3.6
Wyoming	62,101	51,388	10,713	(7)	114	622	----------	320	----------	61,501	600	1.0
District of Columbia	173,519	155,334	18,185	(7)	880	3,106	110	1,758	42,087	156,676	16,843	10.8
Total	25,814,103	22,347,800	3,466,303	349,930	101,074	[10] 172,250	6,030	105,363	9,985,207	[11] 26,545,281	−731,178	−2.8

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[1] This table lists only the number of registrations, licenses, and permits; for financial statement see p. 40,

[2] The first 3 columns show regularly registered motor cars and trucks, eliminating reregistrations, non-resident registrations, etc.

[3] Busses are included with passenger cars, except as noted in next column.

[4] These official cars are exempted from paying regular registration fees and are excluded from registered motor vehicles.

[5] Busses included with trucks.

[6] Classified with trucks.

[7] Not registered.

[8] Registrations during year ending June 30, 1931.

[9] For comparative purposes the figure 252,123 published in 1930 table, which covered a 6-month registration period, is changed to cover a 12-month period, ending June 30, 1930.

[10] Includes 7,859 cars at large owned by United States Government, not allocated to any State.

[11] See note 9 explaining change from previously reported figure.

MOTOR VEHICLE REGISTRATION FEES, LICENSES, PERMITS, FINES, ETC., 1931

[Compiled for calendar or registration year from reports of State authorities]

State [2]	Total gross receipts	Registration receipts [2]					Miscellaneous receipts [2]			Disposition of gross receipts					State [2]
		Motor-car receipts			Other vehicles						For highway purposes				
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles	Dealers, licenses	Chauffeur and operator permits	Other miscellaneous	Collection and administration [3]	State highways	Local roads	State and county road bonds [4]	For other purposes	
Alabama [2]	$3,379,392									$138,793	$1,185,848	$604,674	$1,450,077		Alabama.
Arizona	767,508	$588,321	$340,110	$248,211	$18,421	$1,326	$1,016	$26,370	$132,054	190,092	577,416				Arizona.
Arkansas	3,495,545	3,420,133					35,300	20,750	19,362				[5] 3,495,545		Arkansas.
California	9,763,274	8,531,179	6,184,141	2,347,038	332,081	31,815	42,573	183,484	642,142	1,345,908	3,334,032	3,334,031		[6] $1,749,303	California.
Colorado	1,910,741	1,807,783	1,401,858	405,925	3,025	1,684	24,798	12,480	60,971	163,315	873,713	873,713			Colorado.
Connecticut	8,259,542	6,375,865	4,928,198	1,447,667	20,242	9,670	84,488	1,256,569	512,708	719,000	7,540,542				Connecticut.
Delaware	1,043,173	886,355	639,876	246,479	5,851	1,080	7,078	103,739	39,070		504,993		[7] 538,180		Delaware.
Florida	4,851,968	4,761,161	3,570,994	1,190,167	38,621	5,137	21,212	3,436	22,401	292,439	4,128,094	216,074		[8] 215,361	Florida.
Georgia	4,256,553	4,181,604	3,376,924	804,680	26,958	5,256	25,661	6,876	10,198	211,282	4,045,271				Georgia.
Idaho	1,909,363	1,861,024	1,462,641	398,383	8,990	1,785	20,895	1,514	15,155	57,281	179,652	1,672,430			Idaho.
Illinois	18,426,497	17,411,795	12,932,936	4,478,859	194,692	18,567	72,400	309,713	419,330		10,531,277		7,895,220		Illinois.
Indiana	6,332,101	5,990,293	4,535,504	1,454,789	92,677	4,824	51,650	41,003	151,654	357,172	5,974,929				Indiana.
Iowa	12,539,613	11,812,357	9,968,137	1,844,220	31,497	6,014	68,300	34,113	587,332	373,768	11,789,170			[9] 376,675	Iowa.
Kansas [2]	6,059,910	5,809,511						226,645	23,754	390,900	3,869,010	1,800,000			Kansas.
Kentucky	4,842,108	4,649,430	3,400,792	1,248,638		3,636	24,124	34,639	130,279	469,786	3,756,746	615,576			Kentucky.
Louisiana	4,549,244	4,295,275	3,233,243	1,062,032	148,838	4,190	24,683	68,953	7,305	50,000	4,159,744		339,500		Louisiana.
Maine [10]	3,184,091	2,471,345	1,837,314	634,031	8,984	5,862	50,342	508,453	139,105	337,255	1,362,040		1,333,523	[11] 151,273	Maine.
Maryland	3,497,807	2,663,813	2,304,518	359,295	29,294	9,918	28,089	210,704	555,989	349,781	2,518,421			[12] 629,605	Maryland.
Massachusetts	7,000,306	4,017,860	2,769,618	1,248,242	34,928	4,172	9,999	2,048,608	884,739	1,580,903	4,737,296		682,107		Massachusetts.
Michigan	21,821,290	19,592,989	15,167,237	4,425,752	427,899	12,652	76,946	899,329	811,475	1,064,847	13,041,324	6,000,000	1,082,060	[11] 633,059	Michigan.
Minnesota	10,784,845	10,612,233	8,610,041	2,002,192	53,256	6,192	31,108	53,968	28,088	400,000	4,785,823		[13] 5,536,652	[14] 62,370	Minnesota.
Mississippi [2]	2,421,287									164,322	153,099	2,103,866			Mississippi.
Missouri [2]	10,140,429									527,132	5,331,647		4,281,650		Missouri.
Montana	1,499,492	1,434,255	1,147,545	286,710	262	843	28,310	622		32,144		1,467,348			Montana.
Nebraska	3,742,401	3,598,845	3,049,995	548,850	16,418	2,883	37,439		86,816	114,794	1,088,282	2,539,325			Nebraska.
Nevada	388,327	367,119	272,759	94,360	2,826	311	2,800		15,271	43,089	180,750		164,488		Nevada.
New Hampshire [2]	2,257,459					5,388	25,740	300,062	83,657	201,113	2,056,346				New Hampshire.
New Jersey	15,891,204	11,645,139	7,554,750	4,090,389	115,462	11,476	76,375	3,177,941	864,811	1,125,694	7,229,510	7,310,000		[15] 226,000	New Jersey.
New Mexico	1,248,097	1,207,916	902,830	305,086	5,675	716	7,287		25,565	98,917	689,508	459,672			New Mexico.
New York	41,877,611	36,940,089	26,642,478	10,297,611	178,421	51,920	214,705	3,031,017	1,461,459	[16] 937,211	30,777,359	6,099,597		[17] 4,063,444	New York.
North Carolina [2]	6,164,549									135,000	2,552,000		[18] 3,426,749	[11] 50,800	North Carolina.
North Dakota	1,799,120	1,790,861	1,378,228	412,633		1,158	6,828		273	73,015	1,019,203	706,902			North Dakota.
Ohio	12,818,705	12,055,577	7,036,551	5,019,026	284,623	15,988	68,320	6,835	387,362	496,510	6,040,981	6,281,214			Ohio.
Oklahoma [2]	5,857,107									242,466	2,249,545	3,365,096			Oklahoma.
Oregon	6,940,504	6,236,388	5,295,744	940,644	38,598	6,173	24,688	63,483	571,174	467,478	1,492,236	1,618,257	3,362,533		Oregon.
Pennsylvania	31,607,172	23,408,066	16,083,280	7,324,786	79,975	29,305	252,575	4,626,040	3,211,211	3,297,276	23,262,534		3,436,513	[19] 1,610,849	Pennsylvania.
Rhode Island	2,272,879	1,830,204	1,358,762	471,442	2,016	3,340	14,650	329,572	93,097	266,364	1,932,924	73,591			Rhode Island.
South Carolina	2,790,008	2,505,454	2,020,474	484,980	41,448	2,121	13,285	124,329	103,371	65,231	2,555,093			[20] 169,684	South Carolina.
South Dakota	2,808,172	2,747,209	2,313,799	433,410	7,003	970	17,650		35,080	80,240	1,371,696	1,356,236			South Dakota.
Tennessee [2]	4,580,685									86,321	2,132,670	2,132,670		[11] 229,024	Tennessee.
Texas	13,994,647	12,961,446	9,075,689	3,885,757	365,346	14,454	43,702	130,952	478,747	671,855	4,451,194	8,871,598			Texas.
Utah [2]	828,303									67,697			737,500	[21] 23,106	Utah.
Vermont	2,355,913	2,031,689	1,628,406	403,283		3,411	26,179	248,626	46,008		2,355,913				Vermont.
Virginia	6,159,257	5,896,212	4,970,890	925,322	14,181	8,001	60,500	72,750	107,613	241,409	5,730,617			[11] 187,231	Virginia.
Washington	7,623,570	7,034,491	5,497,235	1,537,256	61,457	9,212	69,170	435,491	13,749	584,290	2,183,355	4,420,434		[11] 435,491	Washington.
West Virginia	4,519,763	4,162,001	3,210,870	951,131	5,535	5,180	37,803	96,237	213,007	236,908	882,855		3,400,000		West Virginia.
Wisconsin	11,724,995	11,392,171	8,592,135	2,800,036	62,705	12,849	91,127	19,994	146,149	800,046	3,557,461	6,121,321	1,246,167		Wisconsin.
Wyoming [2]	727,667										561,667		166,000		Wyoming.
District of Columbia	623,460	193,330	167,404	25,926		880	1,758	126,261	301,231	139,560				[22] 736,422	District of Columbia.
Detailed totals	298,425,321	271,178,788	(23)	(23)	2,758,205	320,359	1,821,553	18,841,558	13,438,762						Detailed totals.
Grand totals	344,337,654									19,688,604	200,733,786	70,043,625	42,574,464	11,549,697	Grand totals.

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[1] Financial data only on this table: For number of registrations, etc., see p. 39.

[2] 10 States thus noted do not report complete details, and receipts are not included in "detailed totals."

[3] States for which no figures are shown make state appropriations for administration.

[4] Payments on State bonds except as noted.

[5] Includes payments of $2,239,615 on county road bonds.

[6] Includes $46,310 reserve for refunds, remainder for State highway patrol.

[7] Includes payments of $349,355 on county road bonds.

[8] For county school fund.

[9] For county general funds.

[10] New law April, 1931, created a State highway fund from which appropriations are made. The disposition shown is on old basis from actual expenditures and is set up to compare with previous year.

[11] State highway patrol.

[12] For Baltimore streets.

[13] Includes $5,154,152 payments on county road bonds.

[14] For State general fund.

[15] Includes $201,000 for free bridge commission and $25,000 for traffic commission.

[16] Excludes $1,827,515 appropriation from State general fund.

[17] For New York City general fund.

[18] Includes $971,749 payments on county road bonds.

[19] State highway patrol, $1,070,019, and payment of gasoline tax collection cost, $540,830.

[20] Includes $38,054 reserve fund, remainder for State highway patrol.

[21] Refund reserve.

[22] For signals and streets as appropriated by Congress.

[23] Only 37 States and District of Columbia separate motor vehicle receipts. The detailed totals for these States are as follows: Passenger car and bus registration receipts, $194,863,906, and truck and tractor receipts, $67,085,238.

ROAD PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.

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MISCELLANEOUS PUBLICATION

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TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927.)

Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States. (1930.)

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Vol. 5, No. 19, D- 3 . . Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6 . . A New Penetration Needle for Use in Testing Bituminous Materials.
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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MARCH 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$				$	$				$	
Alabama	2.379.5	2.008.496.70	982.352.29	88.4		88.4						5.455.698.02	Alabama
Arizona	1.198.6	2.461.045.74	1.732.496.12	148.5	21.6	170.1	54.912.91	41.343.92	5.4		5.4	1.501.293.55	Arizona
Arkansas	1,940.3	3.089.330.65	1.445.224.82	73.8	61.1	134.9	362.032.53	181.016.26	17.3		17.3	2.065.661.91	Arkansas
California	2.291.6	10.179.792.17	4.695.286.79	215.2	38.7	253.9	1.082.442.40	581.339.71	33.8		33.8	1.184.664.55	California
Colorado	1.544.4	4.723.831.84	2.435.540.30	207.4	59.4	266.8	324.036.30	156.005.25	14.2		14.2	2.064.397.14	Colorado
Connecticut	288.5	3.084.244.77	1.151.757.90	22.9		22.9						713.252.14	Connecticut
Delaware	361.6	513.658.00	256.829.00	22.6		22.6	205.355.25	102.677.62	11.3	2.5	13.8	290.727.07	Delaware
Florida	617.5	4.244.486.83	1.941.328.68	111.3		111.3						2.995.837.56	Florida
Georgia	3.075.7	5.765.219.67	2.669.967.14	118.9	155.1	274.0	416.776.79	208.388.37	24.1	3.4	27.5	1.998.394.93	Georgia
Idaho	1.485.3	1.836.334.71	1.065.492.50	89.9	80.3	170.2	423.250.54	255.284.28	26.3	8.8	35.1	1.261.340.62	Idaho
Illinois	2.625.9	24.420.578.64	11.452.862.33	730.8	28.7	759.5	4.029.140.06	1.636.075.76	124.9	12.8	137.7	1.964.464.74	Illinois
Indiana	1.788.4	7.848.719.10	3.853.212.26	208.5		208.5	5.027.471.13	2,390.934.64	204.3	7.3	211.6	551.260.01	Indiana
Iowa	3.390.6						3.276.741.46	1.523.662.43	155.7	42.2	197.9	1.276.142.77	Iowa
Kansas	3.681.9	3.472.112.65	1.633.968.53	204.4	14.5	218.9	3.083.558.94	1.512.148.59	175.9	121.6	297.5	1.201.185.84	Kansas
Kentucky	1.848.5	2.908.041.18	1.347.135.94	161.0	4.0	165.0	452.589.05	223.754.67	8.3	34.8	43.1	1.710.447.10	Kentucky
Louisiana	1.541.7	7.160.318.63	2.775.796.03	112.3	10.6	122.9	956.563.32	478.281.66	.5		.5	1.263.322.55	Louisiana
Maine	706.0	1.804.346.15	789.080.76	31.4		31.4	1.999.657.70	823.804.29	63.5		63.5	513.407.53	Maine
Maryland	776.8	269.787.15	63.776.89	6.3		6.3	586.909.20	292.571.66	25.7		25.7	627.960.27	Maryland
Massachusetts	814.8	7.521.359.24	3.084.570.72	66.8	.2	67.0						688.428.07	Massachusetts
Michigan	1.986.7	9.930.797.55	4.495.918.60	403.0	62.5	465.5	550.262.90	183.660.37	25.3	6.1	31.4	3.105.104.38	Michigan
Minnesota	4.179.6	2.869.940.52	975.961.40	48.3	77.5	125.8	7.343.177.40	2.375.809.81	165.2	182.2	347.4	476.538.62	Minnesota
Mississippi	1.792.7	4.130.076.48	2.028.622.40	169.8	77.6	247.4	103.524.13	51.762.05	9.5		9.5	5.327,002.36	Mississippi
Missouri	2.961.3	2.860.763.74	1.200.594.50	65.6	3.4	69.0	4.294.187.18	1.910.628.60	133.5	53.8	187.3	1.613.641.14	Missouri
Montana	2.673.0	4.574.097.77	2.571.441.21	392.4	17.6	410.0	454.085.69	253.059.57	61.4	.5	61.9	3.483.976.64	Montana
Nebraska	4.145.7	4.960.841.08	2.426.721.22	207.1	76.6	283.7	341.832.91	170.752.29	13.9		13.9	2.584.026.10	Nebraska
Nevada	1.342.3	1.329.619.67	1.025.174.22	38.7	84.9	123.6	590.797.54	524.748.49		55.6	55.6	753.719.43	Nevada
New Hampshire	418.4	636.114.46	246.912.44	12.1	2.5	14.6						553.268.42	New Hampshire
New Jersey	603.3	3.905.196.02	1.538.311.71	38.3	.6	38.9						1.592.129.15	New Jersey
New Mexico	2.192.0	2.356.622.06	1.527.394.54	122.2	16.5	138.7	1.053.109.73	670.732.74	41.1	7.7	48.8	862.054.22	New Mexico
New York	3.262.5	11.445.100.00	5.288.075.00	267.8		267.8	5.214.649.00	2.035.494.50	105.5		105.5	3.689.746.95	New York
North Carolina	2.222.8	1.311.744.48	632.975.21	68.9		68.9	170.890.42	85.445.20	11.3	4.0	15.3	3.830.373.80	North Carolina
North Dakota	5.117.4	1.833.348.18	921.286.74	170.4	171.9	342.3	1.421.317.33	726.174.49	176.3	259.5	435.8	2.090.781.50	North Dakota
Ohio	2.846.1	6.559.165.56	2.289.708.34	98.6	18.3	116.9	950.900.00	356.700.06	16.4	10.0	26.4	3,978.513.51	Ohio
Oklahoma	2.230.1	4.483.089.78	2.137.304.78	112.5	72.0	184.5	1.044.116.66	516.631.00	83.6		83.6	2.049.885.75	Oklahoma
Oregon	1,529.0	3.900.721.84	2.147.345.18	102.0	47.1	149.1	617.548.03	381,618.92	20.1	5.6	25.7	1.514.493.08	Oregon
Pennsylvania	3.012.6	1.905.156.01	773.276.79	45.3		45.3	2.200.751.13	936.470.32	66.5		66.5	4.234.834.04	Pennsylvania
Rhode Island	257.5	809.280.68	400.528.17	16.6		16.6	81.305.62	40.652.81		2.2	2.2	338.736.37	Rhode Island
South Carolina	2,008.6	2.477.211.20	1.054.161.02	56.9	37.3	94.2						1.428.293.19	South Carolina
South Dakota	4.013.6	3.294.275.28	1.840.552.02	195.7	187.1	382.8	559.170.30	374.189.03	102.4		102.4	1.099.884.78	South Dakota
Tennessee	1.675.0	495.757.91	245.741.70	16.4	4.4	20.8	936.864.17	452.877.48	35.8	6.3	42.1	3.629.666.12	Tennessee
Texas	7.577.1	15.000.607.30	6.795.296.07	770.5	180.3	950.8	2.257.585.26	1.015.744.04	148.6	49.9	198.5	5.848.160.10	Texas
Utah	1.218.5	690.137.33	482.342.89	61.0	.4	61.4	576.880.17	426.866.26	37.1	25.9	63.0	1.142.460.84	Utah
Vermont	339.1	104.832.58	52.345.06	4.3		4.3	239.646.33	112.035.00	7.5		7.5	469.931.27	Vermont
Virginia	1,896.6	2.446.965.75	1.169.634.87	128.8	8.9	137.7	159.197.59	79.598.79	5.4		5.4	2.040.053.01	Virginia
Washington	1.195.3	2.535.960.50	1.174.568.56	84.7		84.7	705.650.41	337.300.00	16.9	4.5	21.4	1.446.042.23	Washington
West Virginia	882.9	1.825.407.92	806.803.60	44.7	10.5	55.2	97.329.39	40.854.05	.6		.6	1.238.995.53	West Virginia
Wisconsin	2.672.7	3.441.720.15	1.367.098.99	131.1	7.8	138.9	364.236.86	126.900.00		12.7	12.7	2.344.444.64	Wisconsin
Wyoming	2.022.3	2.616.975.90	1.563.970.41	194.0	84.1	278.1	560.645.85	362.514.57	17.5	65.0	82.5	595.980.54	Wyoming
Hawaii	65.2	897.832.22	411.665.59	27.9		27.9	386.378.64	278.088.61	7.6		7.6	1.762.851.99	Hawaii
TOTALS	100.697.5	198.941.063.74	93.018.412.23	6.716.0	1.724.0	8.440.0	55.562,478.22	25.234.648.16	2,200.2	984.9	3.185.1	94.453.473.07	TOTALS

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 3 ▽ MAY, 1932

HIGHWAY ENTERING WILSON CANYON IN NEVADA

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PUBLIC ROADS

A Journal of Highway Research

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 3 May, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

HIGHWAY TRAFFIC CAPACITY

By A. N. JOHNSON, Dean, College of Engineering, University of Maryland

ONE of the fundamental factors of highway economics is the traffic capacity of a road. This paper reports a study of the relative traffic capacity of 2-lane, 3-lane, and 4-lane highways. The project was undertaken as a cooperative arrangement between the United States Bureau of Public Roads, the State Roads Commission of Maryland, and the University of Maryland, under the immediate supervision of the writer.

The basic data for this discussion are the result of traffic counts taken during the summers of 1930 and 1931. A tentative report on the work done in 1930 was published in the Proceedings of the Highway Research Board for 1930. It was evident that positive conclusions could not be reached from the data then at hand, and a second series of observations was made in 1931, closely paralleling that of the preceding year. There will be repeated here such general description of the work as is necessary to make this report complete in itself without reference to the tentative report noted above.

In 1930 the field work extended from June 26 to September 1, and in 1931 from July 1 to September 7. Observers in both years were recruited among senior engineering students at the University of Maryland.

TRAFFIC CAPACITY AND CONGESTION DEFINED

For the purposes of this study it was first necessary to develop a definition of traffic capacity. The "working capacity" or "free-moving capacity" of a highway was taken to mean the point at which congestion first becomes apparent. When a road carries only a few vehicles all will move freely and there can be no question of congestion. As the number of vehicles increases there will be reached a point at which some will be delayed because they can not immediately pass slower vehicles ahead of them. This delay indicates congestion.

Beyond the free-moving capacity of a highway the number of vehicles passing in a given time may still increase, but traffic will move with more and more restrictions. The individual driver will have less and less freedom of action, being compelled to follow the vehicles directly ahead of him. The number of vehicles may increase until the rate of flow is at a maximum, when the ultimate capacity of the highway may be said to have been reached. Any attempt to put still more vehicles through will result in serious interference with the movement of traffic, and the number of vehicles passing a given point in a given time will actually decrease because of overcrowding.

The observers were instructed to make note of congestion under substantially the following conditions: Congestion is considered to occur on a road when the number of vehicles reaches a total great enough to fill the road and make turning out impracticable; this condition to last a sufficient length of time to be noticeable, the minimum amount of time being one minute. When congestion occurs, reduction of speed will be noticed, along with the tendency for drivers to crowd one another.

In all further discussion the term "congestion" is used as above defined and "capacity" is understood to mean "working capacity."

The two observers traveled from point to point in a motor car and counted the traffic on roads known to have heavy traffic, endeavoring as far as possible to count during the rush hours on the respective roads. The stretches of highway selected were as free as possible from interference from crossroads or other features that would hinder the free flow of traffic.

During 1930, traffic was counted at 38 different points, scattered between Boston and Washington, 51 separate counts being made. In 1931, 33 stations were occupied, for a total of 56 counts, mostly in northern New Jersey and Pennsylvania because of the comparatively large number of three and four lane roads to be found in this territory. A complete list of the stations is given in Table 3, at the end of the article. A summary of the traffic counts according to geographic location and number of lanes of pavement width is shown in Table 1.

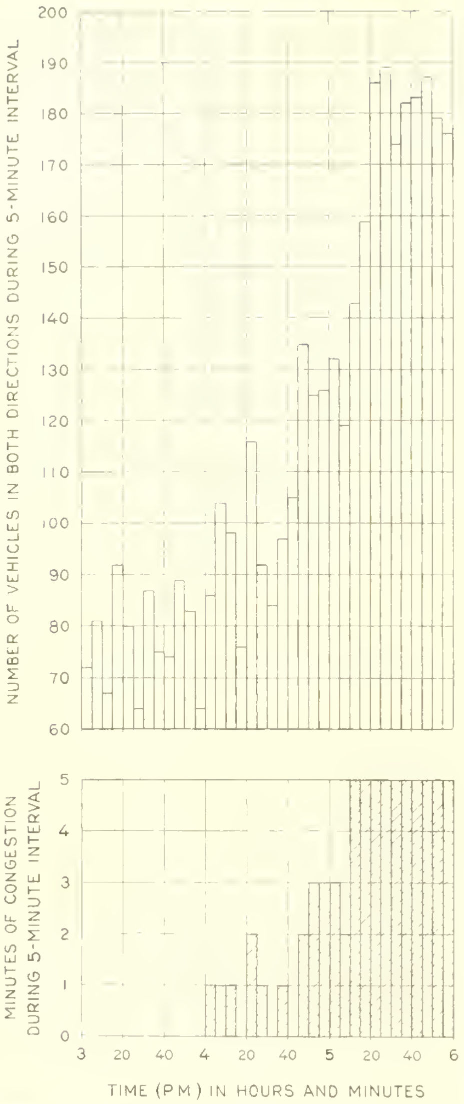
TABLE 1. *Distribution of traffic counts by geographic location and number of lanes*

State	Year	Number of counts: 2-lane roads	3-lane roads	4 lane roads	Total
Maryland	1930	17		4	21
	1931	4		1	5
New Jersey	1930	2	7	2	11
	1931	5	26	5	37
Pennsylvania	1930	3	3		6
	1931	9	6		14
New York	1930	2			2
Connecticut	1930	4		2	6
Massachusetts	1930	2		2	4
Virginia	1930	1			1
Total	1930	31	10	10	51
	1931	18	32	6	56
Total in both years		49	42	16	107

The purpose of the investigation made it necessary to count traffic through peak periods to get as nearly as possible the maximum traffic conditions. It will be readily appreciated, however, that there were many stations occupied which did not at the time develop sufficiently heavy traffic to approach the critical stage. There were, therefore, a number of counts made each year which had no influence upon the results. The rush hours during which traffic stations were occupied proved to be late afternoon or evening, generally between 3 and 7 o'clock.

CONSISTENCY OF OBSERVERS' JUDGMENT TESTED

One of the questions which arose during the analysis of the data was whether under similar traffic conditions like interpretations would be made by the same or different observers; in other words, whether the concept of congestion as formulated was sufficiently definite to serve as a fixed, objective standard. A study of the traffic counts submitted, making comparison between the conclusions reached by one party of observers and those of the other party under similar conditions showed very satisfactory agreement, indicating that such differences as did occur were due more to some other influence that would affect the traffic than to differing judgment on the part of the observers.



HOUR	TRAFFIC (VEHICLES)	PERCENTAGE IN EACH DIRECTION
3 TO 4	924	50 / 50
4 TO 5	1,243	41 / 59
5 TO 6	2 008	25 / 75

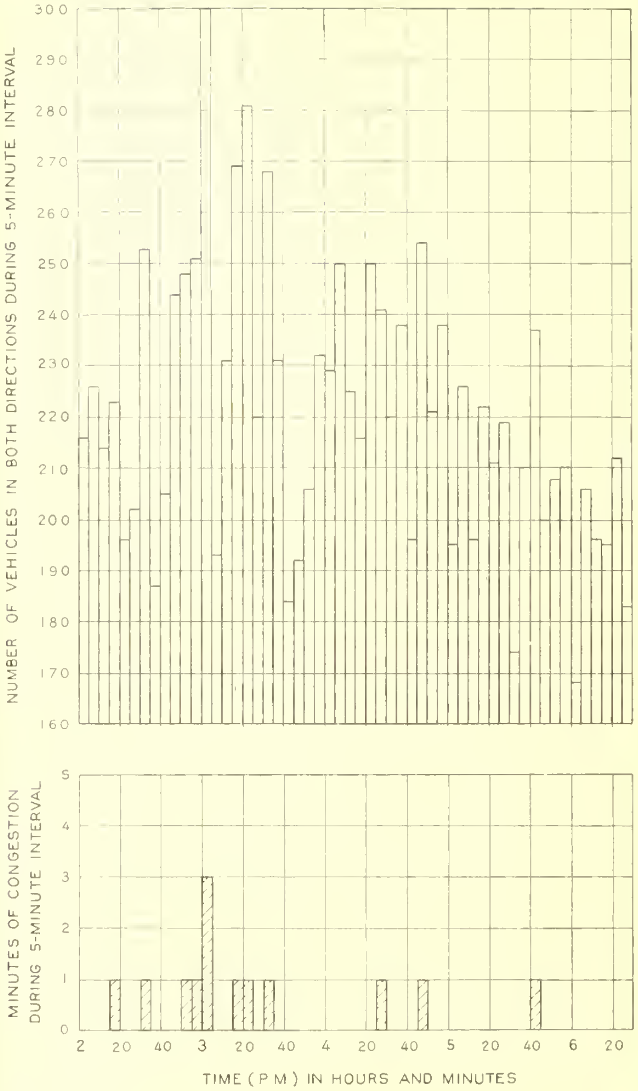
TRUCKS 10 PER HOUR
BUSSES 0
SPEED 25 TO 35 M P H.

Figure 1.—Traffic on 2-Lane Road in Druid Hill Park, Baltimore, Md., July 15, 1930

As a further test of the judgment of the observers, stations that were occupied at the beginning of the season were again occupied at the close, the traffic at these points being about the same. The reports received were similar and indicated that the picture in the observer's mind of a congested condition, as here defined, was reasonably well fixed and precise.

PROCEDURE DESCRIBED

At each station the traffic was recorded by 5-minute intervals, showing the number of vehicles in each direction or in each lane. Passenger automobiles, trucks, and busses were tabulated separately. The tally sheets provided space for indicating when the traffic was running freely and when it was congested. No record of congestion was made unless it extended for a period of one minute. Thus, during some 5-minute intervals there would be but one minute during which congestion occurred; in other instances two minutes, or more, up to the full five minutes.



HOUR	TRAFFIC (VEHICLES)	PERCENTAGE IN EACH DIRECTION
2 TO 3	2,642	28 / 72
3 TO 4	2,805	32 / 68
4 TO 5	2,755	41 / 59
5 TO 6	2,484	53 / 47
6 TO 6:30	1,157	56 / 44

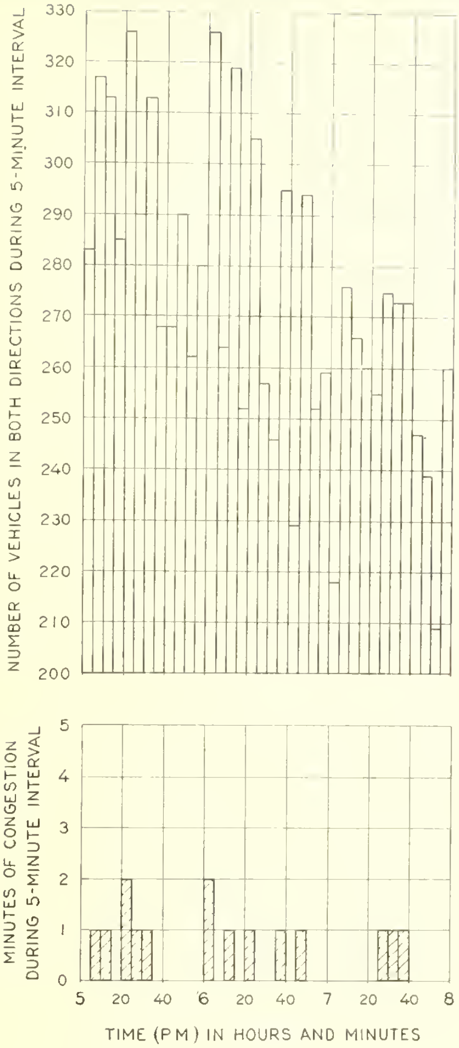
TRUCKS 3¾ PER CENT
BUSSES ½ PER CENT
SPEED 35 TO 40 M.P H

Figure 2.—Traffic on 3-Lane Road, U. S. Route 9, 1 Mile North of South Amboy, N. J., August 1, 1931

The tally sheets were summarized for each count and the data plotted as shown in Figures 1, 2, and 3. These graphs give all the essential facts, such as date, location, actual traffic per hour, estimated average speed, number of trucks and busses, and the number of lanes available for traffic. The ordinates of the graph show the traffic for each 5-minute interval, while the number of minutes of congestion in each interval are shown by the shaded columns in the lower section of the graph.

Figure 1 gives the record of a traffic count on a 2-lane road by 5-minute intervals during the hours between 3 and 6 p. m. The maximum hourly traffic (5 to 6) was 2,008 vehicles, although the maximum rate per hour during a 5-minute period within this same hour was 2,268. This was the maximum hourly traffic observed on any 2-lane road and was about 89 per cent of the hourly rate of the maximum 5-minute interval.

Figure 2 shows a similar record for a 3-lane road where the actual hourly maximum was 2,805 vehicles.



HOUR	TRAFFIC (VEHICLES)	PERCENTAGE IN EACH DIRECTION
5 TO 6	3,496	70 / 30
6 TO 7	3,305	70 / 30
7 TO 8	3,068	78 / 22

TRUCKS 3½ PER CENT
BUSSES 0
SPEED 35 TO 40 M.P.H

Figure 3.—Traffic on 4-Lane Road, U. S. Routes 1 and 9, 15.5 Miles South of Jersey City, N. J.

The maximum hourly rate for a 5-minute interval was 3,600, the total traffic for the hour being about 78 per cent of the maximum rate based on a 5-minute count. This was the maximum hourly traffic observed on any 3-lane road.

Figure 3, for a 4-lane road, shows a maximum hourly traffic of 3,496, while the maximum rate per hour for a 5-minute interval was 3,912, the actual hourly traffic being about 89 per cent of the maximum 5-minute interval. This was the maximum hourly traffic observed on any 4-lane road.

In the analysis of the data to determine the point of incipient congestion, all 5-minute counts were assembled without regard for location or hour of day, each count being treated as a separate statistical unit. They were first grouped according to the major classification of 2-lane, 3-lane, and 4-lane roads. Each of these larger groups was then subdivided according to the proportion of traffic moving in each direction, viz., 50 per cent in one direction (approximately equal in both directions), 60 per cent in one direction (40 per cent in the opposite direction), 70 per cent and 80 per cent in one direction. There were then 12 groups into which the 5-minute counts for all stations were divided, and within each group the items were arranged in order from least to greatest. These were plotted as shown in Figures 4 to 7, inclusive, congestion occurring during any part of any 5-minute interval being indicated in the same manner as in Figures 1 to 3.

The point where traffic congestion first appeared was noted in each case. Thus, in Figure 4, which is a plot of the data for 2-lane roads with 50 per cent of the traffic in each direction, congestion is first observed when 80 vehicles pass in a 5-minute interval. The fact that no congestion was reported in this group of observations for any number less than 80 made it unnecessary to plot or to consider further those counts indicating a less amount of traffic. Similarly, in each of the suc-

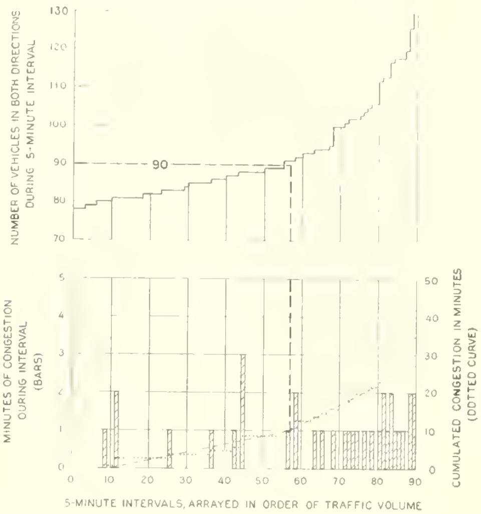


Figure 4.—Determination of Traffic Capacity of 2-Lane Roads, 50 Per Cent of Traffic in One Direction

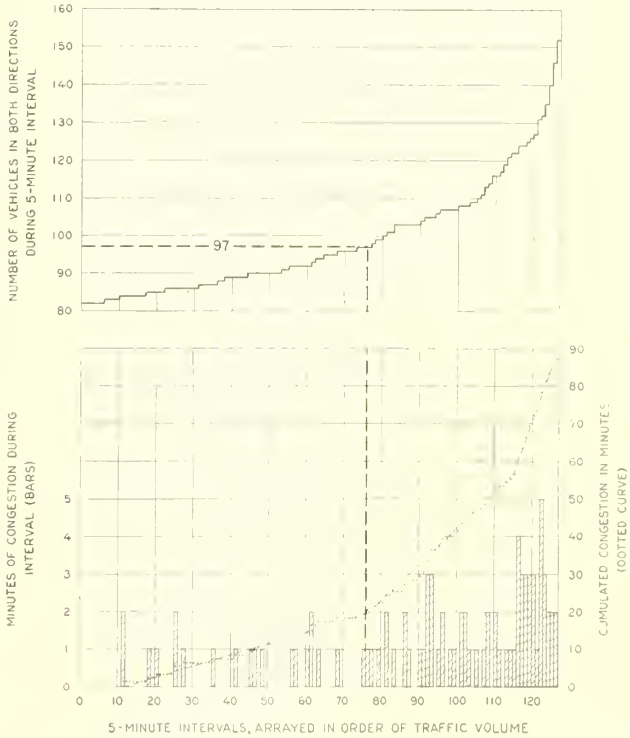


Figure 5.—Determination of Traffic Capacity of 2-Lane Roads, 60 Per Cent of Traffic in One Direction

ceeding diagrams no consideration was given to the observations that recorded traffic insufficient to produce the first instance of congestion.

The critical point to be established in each diagram is the point at which congestion becomes general. This can most easily be determined by inspection of the graphs, which show in nearly every case a fairly definite and sharp transition from a scattered and infrequent occurrence of congestion to a condition in which congestion is usual or normal. The height of the curve at this point indicates the working capacity of the type of highway under consideration.

The nature of the data appears hardly to justify a more elaborate or refined method of analyzing the graphs than that just described. As a check upon the reliability of the inspection method, a cumulative congestion curve was plotted on each diagram. In each case the break in the trend of the cumulative curve, as located by the intersection of straight lines fitted to the separate sections of the curve, coincided almost exactly with the point previously located by inspection.

It will be seen from Figure 4 that the working capacity of two-lane roads with 50 per cent of the traffic in each direction is reached at 90 vehicles per 5-minute interval. Beyond this point, as traffic increases, congestion becomes general for nearly all 5-minute intervals.

For 2-lane roads with 60 per cent of the traffic in one direction, congestion becomes general when traffic exceeds 97 vehicles per 5-minute interval, as shown in Figure 5. When the proportion of traffic is 70 per cent in one direction, congestion appears to become general at 90 vehicles per 5-minute interval, and with 80 per cent in one direction, traffic appears congested at about 105 vehicles per 5-minute interval.

Figures 6 and 7 further illustrate the method of analysis for 3-lane roads with 70 and 80 per cent of the traffic, respectively, in one direction.

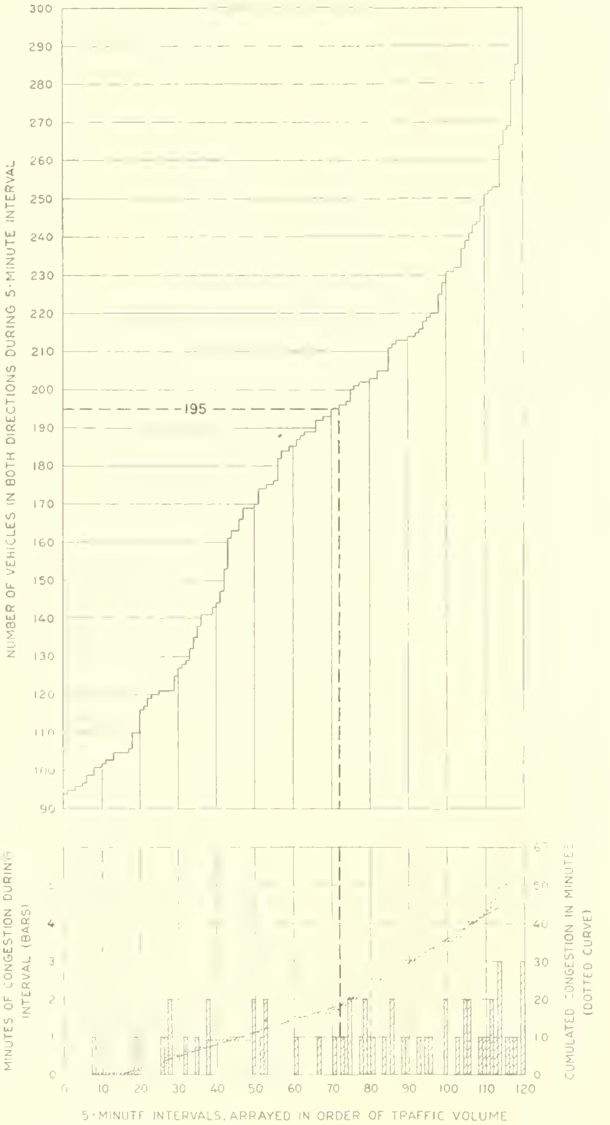


Figure 6.—Determination of Traffic Capacity of 3-Lane Roads, 70 Per Cent of Traffic in One Direction

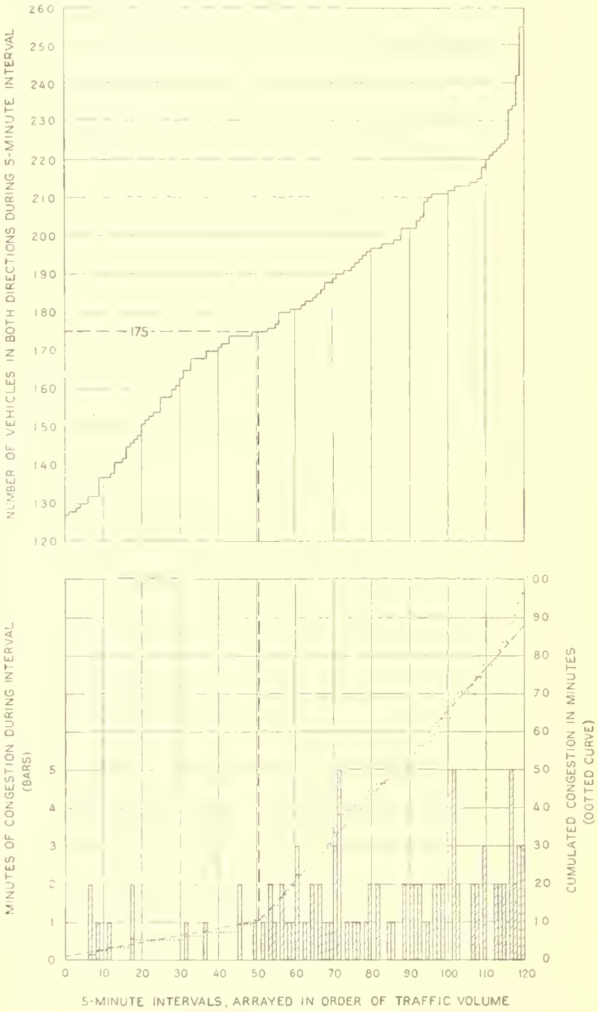


Figure 7.—Determination of Traffic Capacity of 3-Lane Roads, 80 Per Cent of Traffic in One Direction

The evidence brought out by the series of graphs is summarized in Table 2, showing apparent traffic capacity of 2-lane, 3-lane, and 4-lane roads under differing proportions of traffic in the opposing directions.

The values given for practical hourly capacity are based on the 5-minute rate, with reasonable allowance for the fact (demonstrated in figs. 1 to 3) that the maximum rate of traffic during any hour is rarely sustained for more than a small fraction of that hour.

From Table 2 it would appear that the effect of unbalanced traffic on the capacity of 2-lane roads is not marked until 80 per cent of the traffic is in one direction,

TABLE 2.—*Working capacity of 2-lane, 3-lane, and 4-lane highways*

Number of lanes	Vehicles per 5-minute interval					Practical hourly capacity (vehicles)
	Percentage of traffic in one direction				Average	
	50	60	70	80		
2	90	97	90	105	97	1,000
3	185	165	195	175	180	2,000
4	[1] 300	[1] 300	290	270	290	3,000

[1] Estimated.

when the capacity rises considerably above the average. On a 3-lane road the effect is not so definite, but the maximum capacity is reached when about 70 per cent of the traffic is in one direction.

The results for the 4-lane road are even less conclusive, as there was no congestion noted when the traffic was 50 or 60 per cent in one direction, although as many as 273 vehicles were counted during one 5-minute interval. The occasions when 4-lane roads were seen working to capacity were rare. With the traffic 70 per cent in one direction congestion was reported when 290 vehicles passed in five minutes, and when the traffic was 80 per cent in one direction congestion occurred with a count of 270 vehicles. These figures would indicate that, as the traffic becomes more unbalanced, the 4-lane road becomes less efficient.

CONCLUSIONS

The influence of the proportionate amount of traffic in one direction is not marked on 2-lane roads until the fraction increases to 80 per cent or more, when a greater volume of traffic is carried without congestion. The average working capacity for 2-lane roads is approximately 95 per 5-minute interval, or 1,000 per hour.

Three-lane roads appear to operate to slightly better advantage when 70 per cent of the traffic is in one direction. The average working capacity is approximately 180 per 5-minute interval or 2,000 per hour.

Four-lane roads (estimating the capacity as 300 vehicles in five minutes when traffic is 50 or 60 per cent in one direction) have an average working capacity of 290 vehicles per 5-minute interval.

These values give a ratio for 2-lane, 3-lane, and 4-lane roads of approximately 1:2:3. That is, the traffic capacity of a 3-lane road is twice that of a 2-lane road, and the 4-lane road has a capacity of at least three times that of the 2-lane road and 50 per cent greater than that of the 3-lane road.

The addition of one lane to a 2-lane road increases its width 50 per cent and its capacity 100 per cent. Addition of two lanes increases the width by 100 per cent and the capacity by 200 per cent. In other words, doubling the width of a 2-lane highway triples its capacity.

It should be clearly understood and emphasized that this study relates to traffic capacity only. No consideration has here been given to the relative safety of 2-lane, 3-lane, and 4-lane design in highways under varying volumes of traffic. There seems to be very general agreement among those who have observed the operation of 3-lane roads that as traffic increases the hazards increase in a greater ratio than in the case of the 2-lane or the 4-lane roads, but this conclusion must rest upon research of an entirely different nature from that here reported.

TABLE 3.—*List of stations occupied for traffic counts*

MARYLAND

Name of road	Location	Number of lanes	Date	Hours
Baltimore-Washington Blvd., U. S. Route 1	100 yds. S. of S. E. Branch, Anacostia River, Bladensburg.	2	June 27, 1930 July 19, 1930 Aug. 27, 1930	3 to 6 p. m. 11 a. m. to 3 p. m. 3 to 6 p. m.
Do	College Park Experiment Station.	2	June 26, 1930	Do.
Do	1 mi. N. of Laurel	4	July 19, 1931	2 to 4 p. m.
Do	S. of rd. to Dorsey	4	June 28, 1930 Aug. 30, 1930 Aug. 31, 1930 Sept. 1, 1930	3 to 6 p. m. 1 to 4 p. m. 4 to 7 p. m. Do.
Baltimore-Philadelphia Rd., U. S. Route 1.	1 mi. N. of Conowingo.	2	June 29, 1930 July 23, 1931	3 to 6 p. m. 4 to 6 p. m.
Baltimore-Philadelphia Rd., U. S. Route 40.	1 mi. N. of Baltimore city line.	2	July 18, 1930	4 to 7 p. m.
Do	1 mi. S. of Havre de Grace.	2	July 6, 1930 July 8, 1931	2 to 5 p. m. 3 to 6 p. m.
Do	1½ mi. N. of Baltimore city line.	2	Aug. 19, 1930	4 to 6 p. m.
Baltimore-Annapolis Blvd.	3 mi. S. of Glenhurnie.	2	July 20, 1930 Aug 24, 1930	1 to 4 p. m. Do.
Do	Opposite Brooklyn ball park.	2	July 17, 1930 Aug. 25, 1930	4 to 7 p. m. 4 to 6 p. m.
Harford Rd	10 mi. N. of Baltimore.	2	July 22, 1931	Do.
Frederick Rd	3½ mi. E. of Ellicott City.	2	Aug. 12, 1930	Do.
Druid Hill Park Rd., Baltimore.	Opposite Flower Garden.	2	July 15, 1930	3 to 6 p. m.
Charles St. Ave	2 mi. S. of Towson	2	Aug. 20, 1930	4 to 6 p. m.
Reistertown Rd	1 mi. W. of Baltimore city line.	2	Aug. 22, 1930	Do.
Rhode Island Ave. Ext., Mt. Rainier.	William St	2	July 1, 1931	2.45 to 5.45 p. m.
Defense Highway, U. S. Route 50.	¼ mi. E. of rd. to Landover.	2	Aug. 17, 1931	4 to 6 p. m.

NEW JERSEY

Name of road	Location	Number of lanes	Date	Hours
White Horse Pike, U. S. Route 30.	4 mi. E. of Haddon Heights.	3	July 5, 1930 July 4, 1931 July 25, 1931	6 to 8 p. m. 10 a. m. to 1. p. m. 4 to 6 p. m.
Do	9 mi. of E. of Berlin	3	July 18, 1931	Do.
Do	10 mi. W. of Atlantic City.	3	July 12, 1930 July 13, 1930 July 13, 1930 July 4, 1931 July 5, 1931 July 26, 1931 Aug. 9, 1931 Aug. 29, 1931 Aug. 30, 1931	3 to 6 p. m. 10 a. m. to 1 p. m. 3 to 6 p. m. 5 to 8.30 p. m. 5 to 9 p. m. 4.30 to 8.30 p. m. 5 to 8.30 p. m. 3 to 6 p. m. 6 to 9 p. m.
Do	7 mi. W. of Atlantic City.	4	Aug. 9, 1930 Aug. 10, 1930	4 to 6 p. m. 4 to 8 p. m.
Do	6 mi. W. of Atlantic City.	4	July 5, 1931	10.15 a. m. to 1.30 p. m.
Shore Road, U. S. Route 9	1 mi. S. of Woodbridge.	3	Sept. 6, 1931	5 to 8 p. m.
Do	1 mi. N. of South Amboy.	3	July 9, 1930 Aug. 7, 1930 July 11, 1931 Aug. 1, 1931 Aug. 15, 1931 Sept. 4, 1931 Sept. 5, 1931	4 to 6 p. m. 4 to 8 p. m. 3 to 7 p. m. 2 to 6.30 p. m. 3 to 7 p. m. 5 to 8 p. m. 6 to 8 p. m.
Do	1 mi. S. of South Amboy.	2	Aug. 11, 1931 Aug. 14, 1931	5 to 7 p. m. 4 to 7 p. m.
South Amboy-Eatontown Rd., N. J. Route 35.	5 mi. S. of South Amboy.	3	July 12, 1931	Do.
Do	1 mi. N. of Eatontown.	3	July 11, 1930	3 to 6 p. m.
N. J. Route 33	1 mi. W. of Ocean Grove.	2	Aug. 8, 1930	4 to 6 p. m.
Point Pleasant-Eatontown Rd. N. J. Route 35.	1 mi. S. of Eatontown.	3	July 10, 1931	Do.
Atlantic City-Pleasantville Rd., U. S. Route 40.	½ mi. W of Atlantic City.	3	July 9, 1931 July 27, 1931	3 to 6 p. m. 2 to 5 p. m.
Ocean City - Cape May Rd., N. J. Route 4.	10 mi. N. of Cape May.	2	Aug. 8, 1931	3 to 6 p .m.
Broad St., U. S. Route 130.	3 mi. E. of Trenton	3	Sept. 1, 1931	5 to 7 p. m.
Trenton-Jersey City Rd., U. S. Route I	1½ mi. S of Cloverleaf intersection (Woodbridge).	3	July 14, 1931	3 to 5 p. m.
Trenton Jersey City Rd., U. S. Routes 9 and 1.	15.5 mi. S. of Jersey City.	4	Aug. 2, 1931 Aug. 16, 1931 Sept. 5, 1931 Sept. 7, 1931	5 to 8.20 p. m. 3 to 8 p. m. 3 to 5 p. m. 5 to 8 p. m.
Jersey City-Albany Rd., U. S. Route 9W.	2 mi. S of Alpine	3	July 15, 1931 July 16, 1931 July 31, 1931 Aug. 13, 1931	5.15 to 9 p. m. 8 to 10 p. m. 5 to 8 p. m 6 to 8 p. m.

Table 3. *List of stations occupied for traffic counts*—Continued

Name of road	Location	Number of lanes	Date	Hours
NEW JERSEY—Continued				
Westfield-Dunellen Rd., U. S. Route 22	1 mi. W. of Westfield.	2	July 10, 1930	3 to 6 p. m.
			July 13, 1931	5 to 7 p. m.
Somerville-N. Plainfield Rd., N. J. Route 29.	1 mi. N. of North Plainfield.	3	July 17, 1931	4 to 6 p. m.
Dover-Denville Rd., N. J. Route 6.	1 mi. E. of Dover	2	Aug. 12, 1931	Do.
PENNSYLVANIA				
Baltimore-Philadelphia Rd., U. S. Route 1.	1 mi. S. of Clifton Heights.	2	July 1, 1930	3 to 6 p. m.
			Aug. 3, 1931	6 to 9 p. m.
Do	Memorial Bridge, 3 mi. S. of Swarthmore.	2	July 2, 1931	4.20 to 8 p. m.
Roosevelt Blvd., U. S. Route 1.	S. of Frankfort Ave.	3	July 3, 1930	3 to 6 p. m.
			Aug. 23, 1931	3 to 5 p. m.
Do	N. of Vankirk St.	3	Aug. 28, 1931	4 to 6 p. m.
Philadelphia-Trenton Rd., U. S. Route 1.	1 mi. N. of Philadelphia city line.	2	Aug. 17, 1931	3 to 5 p. m.
			Aug. 22, 1931	3 to 6 p. m.
Do	4 mi. S. of Trenton	2	Sept. 2, 1931	4 to 6 p. m.
River Drive, Philadelphia.	Fairmount Park-Ormiston Valley.	3	July 2, 1930	3 to 6 p. m.
			July 3, 1931	Do.
			July 28, 1931	4.30 to 6.30 p. m.
			Aug. 27, 1931	3 to 6 p. m.
City Ave., Philadelphia, U. S. Route 1.	W. of Schuylkill River.	2	July 5, 1930	1 to 4 p. m.
			July 6, 1931	3 to 5 p. m.
			July 24, 1931	3 to 6.30 p. m.
			Aug. 24, 1931	4 to 6 p. m.
Old York Rd., U. S. Route 611.	2 mi. S. of Willow Grove.	2	July 4, 1930	2 to 6 p. m.
			July 29, 1931	4 to 6 p. m.
Susquehanna Trail, U. S. Route 22.	3 mi. N. of Harrisburg.	3	June 30, 1930	3 to 6 p. m.
			Aug. 25, 1931	3 to 4 p. m.
CONNECTICUT				
Boston Post Rd., U. S. Route 1.	Greenwich	4	Aug. 6, 1930	4 to 6 p. m.
Do	Washington Bridge Milford town line.	4	July 27, 1930	4 to 8 p. m.
Do	½ mi. W. of Milford.	2	July 25, 1930	4 to 7 p. m.
			Aug. 5, 1930	Do.
New Haven Ave.	Woodmont	2	July 26, 1930	1 to 4 p. m.
U. S. Route 5.	S. of Hartford city line.	2	July 28, 1930	4 to 6 p. m.
NEW YORK				
U. S. Route 9W	½ mi. S. of West Point.	2	July 22, 1930	4 to 6 p. m.
U. S. Route 9	1 mi. N. of Tarrytown.	2	July 24, 1930	3 to 6 p. m.
MASSACHUSETTS				
Name of road	Location	Number of lanes	Date	Hours
U. S. Route 1	1 mi. N. of Dedham	2	Aug. 1, 1930	4 to 7 p. m.
Massachusetts 3A	4 mi. E. of Quincy	4	Aug. 2, 1930	Do.
			Aug. 3, 1930	Do.
U. S. Route 20	1 mi. W. of South Sudbury.	2	July 31, 1930	Do.
VIRGINIA				
Washington-Alexandria Rd., U. S. Route 1.	2 mi. N. of Alexandria.	2	Aug. 16, 1930	2 to 5 p. m.

CORRECTION

In the April issue of Public Roads there was printed, on page 40, a table entitled "Motor-vehicle registration fees, licenses, permits, fines, etc., 1931." It has been found necessary to revise the figure given in the last column of this table for the District of Columbia, and the resulting grand total. The correct figures are as follows:

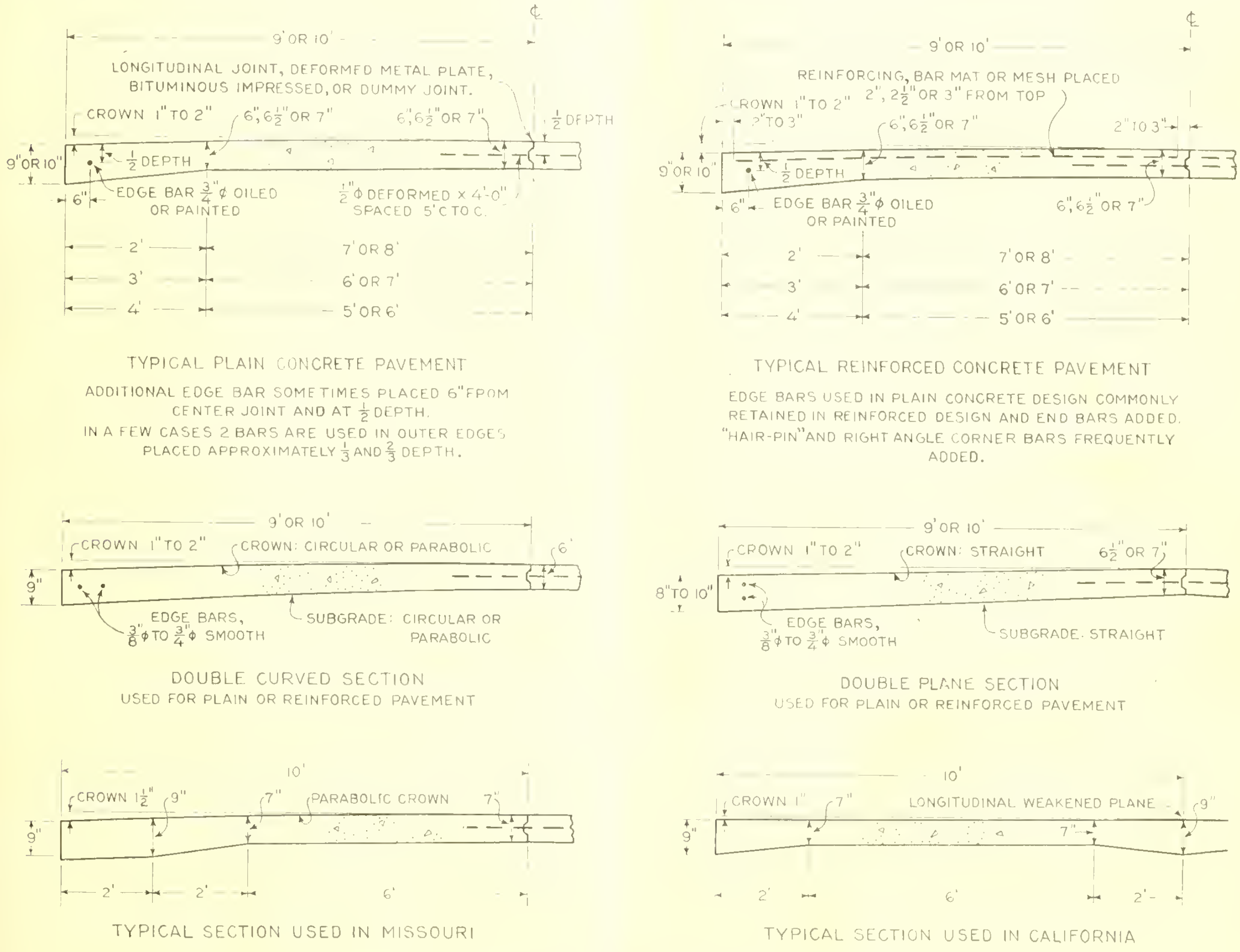
Disposition of gross receipts

	For other purposes
District of Columbia	$483, 900
Grand total	11, 297, 175

CONCRETE PAVEMENT DESIGN FEATURES, 1931

Reported by R. D. BROWN, Senior Highway Engineer, Division of Design, U. S. Bureau of Public Roads

OF THE total of 88,713 miles of roads improved with Federal aid up to June 30, 1931, 28,009 miles were paved with Portland cement concrete. This represents the greatest mileage of any type of improvement or group of related types except untreated gravel, of which 28,646 miles have been constructed. The proportion of concrete pavement to other types constructed with Federal aid during the calendar year 1931 is even greater, clearly indicating the increasing importance of concrete pavement.


9' OR 10'
LONGITUDINAL JOINT, DEFORMED METAL PLATE, BITUMINOUS IMPRESSED, OR DUMMY JOINT.
CROWN 1" TO 2"
6", 6½" OR 7"
6", 6½" OR 7"
½ DEPTH
9" OR 10"
½ DEPTH
6"
EDGE BAR ¾" ϕ OILED OR PAINTED
½" ϕ DEFORMED x 4'-0" SPACED 5' C TO C.
2'
7' OR 8'
3'
6' OR 7'
4'
5' OR 6'

TYPICAL PLAIN CONCRETE PAVEMENT

ADDITIONAL EDGE BAR SOMETIMES PLACED 6" FROM CENTER JOINT AND AT ½ DEPTH.
IN A FEW CASES 2 BARS ARE USED IN OUTER EDGES PLACED APPROXIMATELY ⅓ AND ⅔ DEPTH.

9' OR 10'
REINFORCING, BAR MAT OR MESH PLACED 2", 2½" OR 3" FROM TOP
CROWN 1" TO 2"
2" TO 3"
6", 6½" OR 7"
2" TO 3"
9" OR 10"
½ DEPTH
6"
EDGE BAR ¾" ϕ OILED OR PAINTED
6", 6½" OR 7"
2'
7' OR 8'
3'
6' OR 7'
4'
5' OR 6'

TYPICAL REINFORCED CONCRETE PAVEMENT

EDGE BARS USED IN PLAIN CONCRETE DESIGN COMMONLY RETAINED IN REINFORCED DESIGN AND END BARS ADDED. "HAIR-PIN" AND RIGHT ANGLE CORNER BARS FREQUENTLY ADDED.

9' OR 10'
CROWN 1" TO 2"
CROWN: CIRCULAR OR PARABOLIC
6"
9"
EDGE BARS, ⅜" ϕ TO ¾" ϕ SMOOTH
SUBGRADE: CIRCULAR OR PARABOLIC

DOUBLE CURVED SECTION
USED FOR PLAIN OR REINFORCED PAVEMENT

DOUBLE PLANE SECTION
USED FOR PLAIN OR REINFORCED PAVEMENT

TYPICAL SECTION USED IN MISSOURI

TYPICAL SECTION USED IN CALIFORNIA

FIGURE 1.—TYPICAL CROSS-SECTION DESIGNS

Considerable progress has been made in the past few years toward standardization of the design of concrete pavements. Further progress in this direction is possible in spite of the acknowledged fact that complete uniformity of design is impracticable because of varied conditions of climate, subgrade, and loading in the various States. Starting with only a vague understanding of the structural requirements of cement concrete as a pavement material, intensive research and observation of the earlier designs has pointed the way to rational methods of design affecting not only the shape of the pavement cross section and its dimensions but also the proportions of cement and aggregates, amount and location of steel, and character and location of joints.

A study of concrete pavement designs submitted by the several States during 1931 for use on Federal-aid projects has been made; and in order to facilitate comparison of the different State designs and also comparison with earlier studies,[1] the attached table has been prepared.

The data presented in the table cover the designs for only 43 States, since 5 States either did not submit any projects during 1931 involving the construction of concrete pavements or submitted so small an amount of such work as to be of little value in determining their typical practice.

One important fact brought out by the study is the practically complete acceptance of the 9-foot traffic lane as a minimum width. Highway engineers are almost unanimously agreed upon the desirability of the 10-foot traffic lane and in 20 States it is the standard width. Twenty-two States continue to build 2-lane roads, 18 feet wide, but in all except seven of these States 20-foot pavements are used on their more important routes.

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[1] Public Roads, vol. 8, No. 1, March, 1927, and vol. 10, No. 5, July, 1929.

In all cases where pavements of more than two lanes are built, the width is a multiple of 10 feet.

In the matter of depth of pavement it is found that pavements of uniform depth are still constructed to some extent, particularly in the northeastern States and in connection with rather heavy steel reinforcement. Although there is no question that these designs have proved adequate as to strength there would appear to be grounds to warrant doubt as to the economy of the design. The edge thickness most commonly used on thickened edge designs is 9 inches, although nine States use an edge thickness of 8 inches, one 7½ inches, two 7 inches, and one State occasionally uses a 6-inch uniform depth. Only two States use a double-thickened section. Seventeen States use a 6-inch center depth either as a standard design or for their less important roads. Five States us a 6½-inch center depth, eighteen use 7 inches, and no State uses more than 7 inches at the center except those which use the double-thickened section or uniform 8-inch or 9-inch pavements.

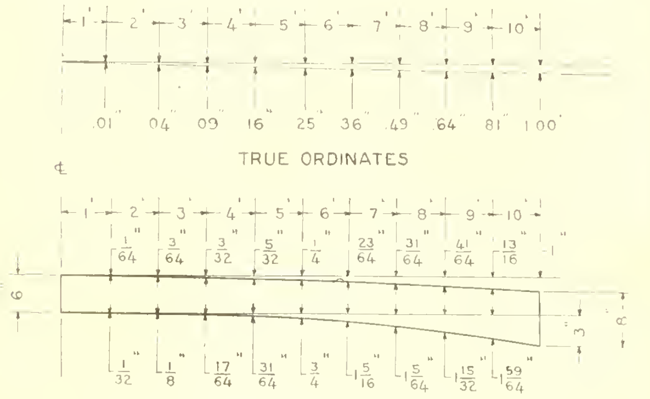


Figure 2. Diagram Showing True Ordinates for Parabolic Crown of 1 Inch in 10 Feet and Practicable Ordinates for Both Crown and Subgrade

The use of a curved subgrade, usually parabolic, in connection with thickened edge designs, has become a more general practice in the past few years because of the greater ease with which a subgrade of this shape is prepared and properly compacted as compared with the angular-shaped subgrade. Ten States now use the curved subgrade with thickened edge designs. One State uses a 2-plane subgrade, the pavement depth increasing uniformly from the center to the edges. The transition distance between edge and center depth in other designs varies from 2 feet to 4 feet. One State uses a uniform edge depth for the outer 2 feet of width, reducing the thickness in the next 2 feet to the center depth.

The reduction in the amount of crown used in concrete pavements in the past five years is notable. In 1926 only two States used a crown as low as 1 inch for a 20-foot pavement and only three used 1 inch for an 18-foot pavement. In 1931, 14 States regularly used a 1-inch crown for a 20-foot pavement and only four use as much as 2 inches. The crown shape is usually parabolic or circular although there is no apparent advantage in this shape over a straight or uniform slope. In case a two-lane pavement with parabolic crown is to be widened by building an additional lane on either side it will be found that this crown shape is objectionable because it produces a break in the pavement slope at the edge of the old pavement unless an unnecessarily heavy crown is used on the new outer lanes.

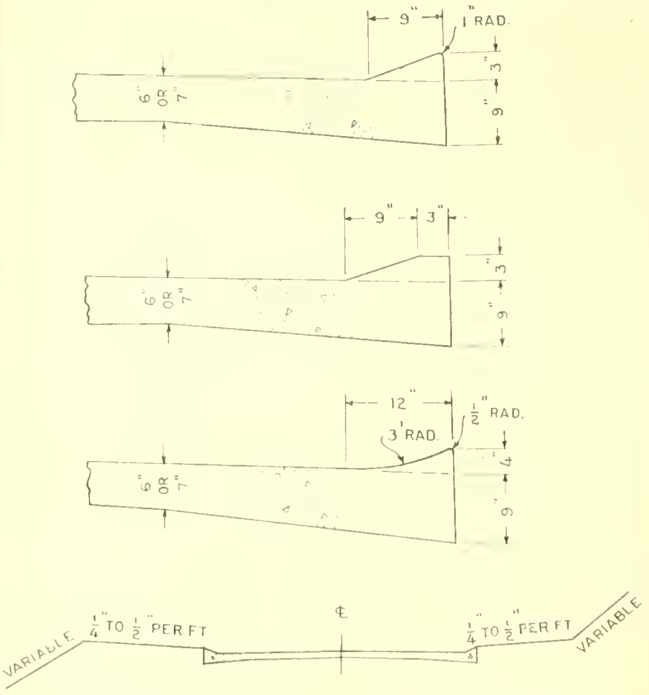


Figure 3. Lip or Sloping Curb Designs

On 4-lane pavements a parabolic crown produces an excessive slope in the outer lanes and a straight crown is much to be preferred. A crown of 3 or 3½ inches for a 40-foot pavement is adequate for all ordinary conditions. Of this total crown 1 to 1½ inches may be used on the inner 10-foot lane and 2 inches in the outer lane. On the Mount Vernon Memorial Highway the normal width of pavement is 40 feet, the center 18 feet of which is crowned to conform to the arc of a circle with a rise of 1 inch in 9 feet. The outer lanes are 11 feet wide, the crown being straight and having a rise of 2½ inches. This crown treatment gives a pleasing appearance and lends itself readily to future widening of the pavement.

In recognition of the public demand for smooth riding qualities and also with a view to reducing impact stresses, the surface trueness requirement has been set at not more than ⅛-inch variation in 10 feet in 11 States. With machine methods of finishing and reasonably close attention to finishing details there does not appear to be any difficulty in meeting this requirement. It is anticipated that many of the States which now permit a variation of ¼-inch in 10 feet will soon adopt a more stringent requirement.

Steel has been used as a reinforcing element in cement concrete for buildings, bridges, and other structures for a long time and quite definite design requirements for such structures have been established. A concrete pavement, however, presents conditions considerably at variance with such structures. The pavement slab receives more or less continuous support from the subgrade but frictional stresses and volumetric changes in the subgrade introduce variable tensile stresses indicating the use of steel. In determining the classification of the various pavement designs as between reinforced and plain, it was considered that a reinforced pavement is one in which the steel is used as a network bonded with the concrete and distributed generally throughout the slab. Marginal bars, end bars, and corner bars when

35
6
21 0

used alone are not considered as reinforcement but when used in addition to a bar mat or mesh reinforcement their weight is included in the tabular data as reinforcement.

Reinforcement is most commonly placed 2 inches from the top of the pavement. Two of the New England States, however, place the reinforcement 2 inches from the bottom and two other New England States use double reinforcement; that is, in both the top and the bottom of the pavement. The severe climatic conditions in these States are undoubtedly accountable for these departures from the general practice. Fifteen States used reinforced designs exclusively, while eight others used the reinforced design only on certain projects. The use of reinforcement is not confined to any geographical area, but most of the States using this design are in the area where frost conditions are rather severe. The smallest amount of steel used in reinforced designs is 25 pounds per 100 square feet and the maximum is 119 pounds. There is a noticeable tendency toward the use of reinforcement for crack control.

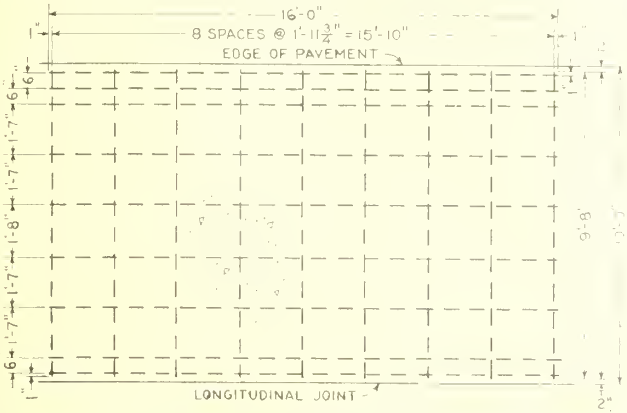


Figure 4.—Typical Bar Mat for Reinforced Pavement; Bars ¼ to ½ Inch Round, Welded, Clipped, or Wired at All Points of Intersection; Mats Lapped 12 to 14 Inches

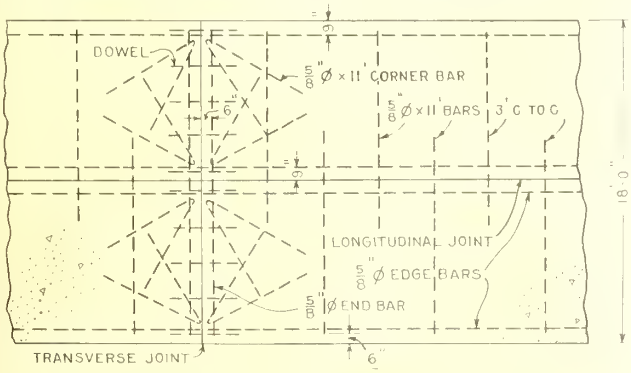
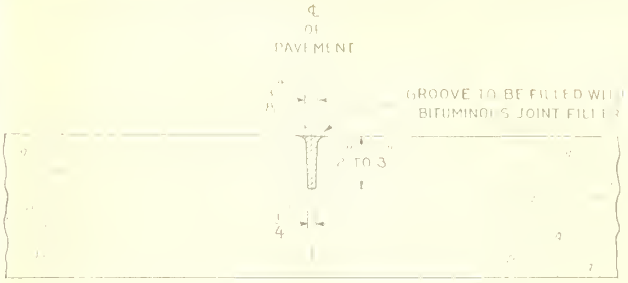


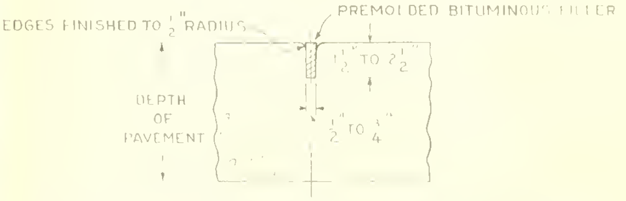
Figure 5.—Design of Semireinforced Slab

In the 28 States submitting plain concrete pavement designs, 13 used varying amounts of steel in the form of edge bars, end bars, and corner bars. In some cases the amount of steel so used exceeds the amount used in reinforced designs in other States.

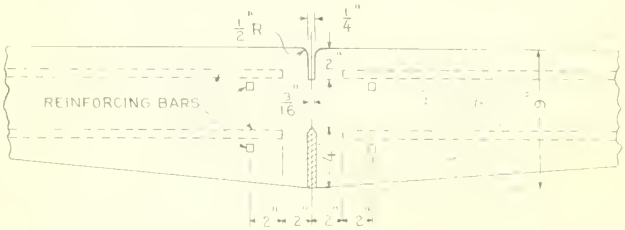
The use of the longitudinal joint is the most universally accepted feature of concrete pavement design. Only two States now build two lane pavements without longitudinal joints. A few years ago it was the standard practice to use a deformed metal plate in the longitudinal joint but the dummy joint or bituminous impressed joint is now permitted as an alternate with the steel plate in 17 States and is used exclusively in four other States. In four States the longitudinal joint is a simple expansion joint without tie bars or dowels. Five States use a construction joint with or without tie bars and 11 States still specify the deformed metal separator.



TRANSVERSE OR LONGITUDINAL WEAKENED PLANE



LONGITUDINAL OR TRANSVERSE BITUMINOUS IMPRESSED JOINT



LONGITUDINAL WEAKENED PLANE USED IN CALIFORNIA

Figure 6.—Typical Designs of Weakened Plane or Dummy Joint

The practice with reference to transverse joints continues to vary greatly among the States, but there is a definite trend toward the use of expansion joints and the use of intermediate dummy contraction joints. Two States use wide expansion joints, 3 inches and 4 inches, spaced at wide intervals, and four States do not provide for any transverse joints except necessary construction joints when the mixer is stopped for 30 minutes or more. The majority of the States use slip dowels across the expansion joints, but no dowels or tie bars across transverse dummy joints. Twenty States permit the use of either premolded expansion joint material or a poured mastic, at the option of the contractor. Four States specify a poured mastic and 15 specify premolded material.

No attempt was made to tabulate data with reference to the mix proportions of the concrete used in the several States for the reason that no simple and satisfactory basis of comparison has been established. Beginning with the construction season of 1930 it has been required that weight batching of aggregates be used on Federal-aid projects. In some States volumetric proportions are converted directly to batch weights for the job aggregates but the principles of a designed mix are more commonly used. It is believed that the requirement of weight batching is well justified as it produces more uniform concrete at no increase in cost and in the States which have adopted the principle of a designed mix considerable economy has resulted from reduction in the cement content while maintaining high strength.

GENERAL FEATURES OF DESIGN OF CROSS SECTION OF CONCRETE PAVEMENTS ON FEDERAL-AID PROJECTS SUBMITTED IN 1931

State	Width	Thickness			Width of thickened edge, in feet	Crown	Maximum surface variation in 10 feet	Steel in reinforced type			Steel in plain type
		Edge	Center	Edge				Bars, pounds per 100 sq. ft.	Mesh, pounds per 100 sq. ft.	Location	
	Feet	*Inches*	*Inches*	*Inches*		*Inches*	*Inches*				
Alabama	18, 20	9	6	9	3	2	⅛		25	3 ins. from top; two ¾-in. round edge bars.	Two ¾-in. edge bars; dowels.
Arizona	18	9	([1])	9	2	1⅛	¼				None.
Arkansas	18, 20	9	6	9	Curved	1½, circular	¼		41 or 49	2½ ins. from top; four ⅝-in. round edge bars.	Four ⅝-in. round edge bars; dowels.
California	20	9	([2])	9	2	1	⅛				Four ½-in. square edge bars; end bars.
Colorado	18, 20	9	6½	9	3	1½, parabolic	¼				None.
Connecticut	20	8	8	8		1, circular	¼	79.41	70.22	2 ins. from bottom [3]	
Delaware	18	8	6	8	Curved	1½ to 2	⅛				Dowels only.
	20	8	8	8							
Florida	20	9	6	9	3	1¼	¼				Do.
Georgia	20	9	6	9	3	1½	¼				Do.
Illinois	18	9	6	9	2	1, circular	⅛				Two ¾-in. edge bars; dowels.
	20	9	7	9	3	do	⅛				Two ⅞-in. edge bars; dowels.
Indiana	18, 20	9	7	9	2½	1¼	¼				Two ¾-in. edge bars; dowels.
Iowa	18	10	7	10	4	2	¼	55		3⅜ ins. from top	
Kansas	18, 20	9	7	9	4	1, parabolic	¼		49	2 ins. from top; edge bars	Two ¾-in. edge bars; ¾-in. end bars.
Kentucky	18	9	6	9	2	1	¼				Dowels only.
	20	9	6½	9	2	1	¼				Two ¾-in. edge bars.
Louisiana	18	8	6	8	Parabolic	1, parabolic	⅛		44	2 ins. from top	Dowels only.
	20	8	7	8	do	1¼, parabolic					
Maine	20	9	7	9	4	1½	¼	119		1½ ins. from top and bottom	
Maryland	16	9	6½	9	Parabolic	1	¼				None.
	18					1⅛					
	20					1¼					
Massachusetts	20	8	8	8		2½	¼	103		2 ins. from bottom	
	30					2¾					
	40					3⅞					
Michigan	20	9	7	9	3	1½	¼	60	60	2 ins. from top	
Minnesota	20	9	7	9	4	1	¼				Six ⅝-in. edge bars; end bars.
Mississippi	20	9	6	9	3	1½	⅛		48.8	2 ins. from top; edge bars	Six ¾-in. edge bars; dowels.
Missouri	20	9	6	9	4	1½, parabolic	¼				Four ¾-in. edge bars; dowels; end bars.
Nebraska	20	9	7	9	3	do	¼	25		2½ ins. from top	
New Hampshire	20	9	6	9	2	1	¼		65.8	2 ins. from top	
New Jersey	20, 40	8	8	8		1½, 2½	⅛	83	78	do	
		9	9	9							
New York	20	8	7	8	10	1¼, straight	¼	78.8	81.1	2½ ins. from top	
		8	8	8							
North Carolina	[4] 18	6	6	6		1½	5/16				Dowels only.
		8	6 or 7	8	Curved						
North Dakota	20	9	7	9	4	1	¼	48.3	57	2 ins. from top	
Ohio	20	9	7	9	2	1¼	¼	51.8	51.4	do	Two ¾-in. round edge bars; dowels.
Oklahoma	20	10	7	10	4	1, parabolic	¼	40.8		3 ins. from top	
Oregon	20	9	7	9	3	1½	¼				Dowels only.
Pennsylvania	18, 20	9	7	9	Curved	1, parabolic	⅛	40 to 44	46	2 ins. from top; corner bars	Six ⅝-in. round edge bars; end and corner bars; dowels.
Rhode Island	20, 40	8	8	8		1½, 4½	¼		45	2 ins. from top; end bars	
South Carolina	18, 20	7½	6	7½	Parabolic	1½	⅛				Dowels only.
South Dakota	20	9	6	9	4	1¼	¼		64.7	2½ ins. from top; edge and end bars.	
Tennessee	18, 20	8	6	8	Parabolic	1	⅛				Dowels only.
Texas	18, 20	9	6	9	4	1, circular	5/16				Four ½-in. round edge bars; end and corner bars.
Utah	18, 20	9	6	9	2	1	¼				Dowels only.
Vermont	18	7	7	7		1, straight	¼	83		2 ins. from top and bottom	
Virginia	18	8	6	8	Curved	1.01	[5] ⅛				None.
Washington	18	9	6½	9	2	1, straight	¼				Dowels only.
		10	7	10							
West Virginia	18	7	7	7		1, parabolic	⅛		43	2 ins. from top	
Wisconsin	20	9	6½	9	4	do	⅛		58	do	Dowels only.

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[1] Two 9-6-9-inch sections, 9 feet wide.
[2] Two 9-7-9-inch sections, 10 feet wide.
[3] One panel (5 feet 2 inches wide) of reinforcement placed 2 inches from top, adjacent to transverse joint on each side.
[4] Six-inch uniform section with stone aggregate corresponds to 8-6-8-inch section with gravel aggregate; 8-6-8-inch section with stone aggregate corresponds to 8-7-8-inch section with gravel aggregate.
[5] In 16 feet.

GENERAL FEATURES OF DESIGN OF CONCRETE PAVEMENTS ON FEDERAL-AID PROJECTS SUBMITTED IN 1931

State	Longitudinal joints			Transverse joints				
	Type	Gage, or width, in inches	Tie bars	Type	Spacing to nearest even foot, in feet	Width *Inches*	Filler	Dowels
Alabama	Steel plate or Bituminous impressed.	16 g. ¾ by 1½	½-in. round, deformed, 5 ft. c. to c. None	Expansion	[1] 30 40	¾ ½	Poured or premolded.	Eight ¾-in. round, 4 ft. long, 1 end fixed.
Arizona	Expansion, premolded.	½	None	do	30	1	Premolded	None
Arkansas	Steel plate or Dummy joint	16 g	½-in. round, deformed, 5 ft. c. to c.	do	50	¾	Poured	Six ½-in. round, 1 end fixed
California	Dummy joint		End bars only	Expansion with dummy joint.[2]	60	½	Premolded	Edge bars only.
Colorado	do		None	Expansion	60	½	do	None.
Connecticut	Expansion	¼	None	do	61	½	Poured or premolded.	None.
Delaware	Steel plate or Dummy joint	16 g	½-in round, 3 ft., 3 ins. c. to c.	do	80 to 85	¾	Premolded	Eight ¾-in., 2 ft. long, 1 end fixed
Florida	Steel plate	18 g	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	do	40	⅝	Poured or premolded.	Ten ½-in round, 4 ft. long, 1 end fixed.
Georgia	do	16 g	½-in round, deformed, 4 ft. long, 4 ft. c. to c.	do	50	¾	do	Eight ⅝-in. round, 4 ft. long, 1 end fixed.
Illinois	do	18 g	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	do	800 to 1,000	4	Poured	None.
Indiana	do	16 g	⅝-in round, deformed, 4 ft. long, 5 ft. c. to c.	Construction	Necessary			Four or six ¾-in., 4 ft. long, 1 end fixed.
Iowa	Steel plate or Dummy joint	18 g	⅝-in. round, smooth, 11 ft long, 3 ft. c. to c.	Expansion [3]	40, 60, 80	1, ¾, ½	Poured or premolded.	Ten ⅝-in. round, 2 ft. long, 1 end fixed.
Kansas	Steel plate, Bituminous impressed or Dummy joint	([4]) ⅜ by 2¾	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	Expansion with dummy joint.[5]	116	1	do	None
Kentucky	Steel plate	16 g	do	Construction	Necessary			None.
Louisiana	do	16 g	do	Dummy joint	40 to 60		Poured or premolded.	Eight ¾-in., 4 ft. long, 1 end fixed
Maine	Construction		None.	Expansion	40	½	Premolded	Ten ¾-in., 3 ft. long.
Maryland	Bituminous impressed; or Dummy joint	⅛ by 2¾	None	Bituminous impressed, or dummy joint.	40		Poured or premolded.	None.
Massachusetts	Construction		½-in. square, deformed, 4 ft. long, 5 ft. c. to c.	Expansion	60	½	do	Eight, twelve, or sixteen ½-in. square, 4 ft. long, 1 end fixed.
Michigan	Steel plate or Dummy joint	16 g	½-in round, 4 ft. long, 20 ins. c. to c.	Expansion	100	1	Premolded	None.
Minnesota	Steel plate or Dummy joint	16 g	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	Expansion with dummy joint [6]	81	½	Poured or premolded.	Eight ¾-in., 2 ft. 6 in. long, 1 end fixed.
Mississippi	Steel plate or Bituminous impressed.	18 g ⅜ by 2½	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	Expansion with dummy joint.[6]	60	¾	do	Eight ¾-in., 4 ft. long, 1 end fixed.
Missouri	Steel plate or Bituminous impressed.	16 g. ⅜ by 2½	do	do [6]	81	1	Premolded	Eight ¾-in. round, 2 ft. 6 ins. long, 1 end fixed.
Nebraska	Steel plate or Bituminous impressed.	18 g ⅜ by 2½	⅝-in. round, smooth, 12 ft. long, 3 ft. c. to c.	Expansion with dummy joint.[5]	100	1	Poured or premolded.	Ten ⅝-in. round, 2 ft. long, 1 end fixed. None.
New Hampshire	Construction		⅝-in. round, deformed, 4 ft. long, 3 ft. c. to c.	Expansion	50	½ to ¾	Premolded	Ten ⅝-in. round, 2 ft. long, greased.
New Jersey	Expansion	½	None	do	34 or 45	½ to ¾	Poured or premolded.	Twelve to twenty-four ¾-in. round, 1 ft. 8 ins. long.
New York	Construction		None	do	78	¾	do	Ten ¾-in. round, 1 ft. 8 ins. long, 1 end fixed.
North Carolina	None			Construction	Necessary			Nine ¾-in. round, 4 ft. long, oiled.
North Dakota	Steel plate or Dummy joint	16 g	½-in. round, deformed, 4 ft. long, 4 ft. c. to c.	Expansion with dummy joint or steel plate.[6]	81	1	Poured or premolded.	Eight ¾-in. round, 2 ft. 6 ins. long, 1 end fixed.
Ohio	Steel plate or Dummy joint	14 g	½-in. round, deformed, 5 ft. long, 5 ft. c. to c.	Expansion with dummy joint.[2]	105 to 120	1	do	Six ¾-in. round, 4 ft. long, 1 end fixed.
Oklahoma	Steel plate or Dummy joint	([4])	½-in. round, deformed, 19 ft. 6 ins. long, 5 ft. c. to c.[7]	[8] Expansion joint Dummy joint	50, 100 50	1	Poured	None.
Oregon	Steel plate	18 g	½-in. round, deformed, 3 ft. long, 4 ft. 6 ins. c. to c.	Expansion with dummy joint.[2]	60	½	Premolded	Four ¾-in. round, 3 ft. long, 1 end fixed.
Pennsylvania	Steel plate or Dummy joint	14 g	½-in. round, deformed, 4 ft. 6 ins. long 5 ft. c. to c.	Expansion	Designed	½	do	None.
Rhode Island	Expansion	¾	None	do	100	¾	do	Eight to sixteen ½-in. round, 2 ft. long, painted.
South Carolina	Steel plate [9]	16 g. to 18 g.	½-in. round, deformed, 4 ft. long, 5 ft. c. to c	do	40	½ to ¾	Poured or premolded.	None.
South Dakota	do	18 g	do	do	40	⅜	Premolded	Eight ¾-in. round, 2 ft. 6 ins. long, 1 end fixed.

[1] For reinforced type, 30-foot spacing, ¾-inch width; for plain type, 40-foot spacing, ½-inch width.
[2] Dummy joints at third points between expansion joints.
[3] Spacing and width of expansion joints dependent on type of aggregate
[4] Gage of metal strip not specified.
[5] Dummy joints at fourth points between expansion joints.
[6] Dummy joint halfway between expansion joints.
[7] Transverse reinforcing bars used as dowels; no short dowels used
[8] Alternate expansion and contraction joints used, Apr. 15 to Oct. 15, at 50-foot spacing; expansion joints at 50-foot spacing, Oct. 15 to Apr. 15
[9] On clay subgrade.

GENERAL FEATURES OF DESIGN OF CONCRETE PAVEMENTS ON FEDERAL-AID PROJECTS SUBMITTED IN 1931—Continued

State	Longitudinal joints			Transverse joints				
	Type	Gage, or width, in inches	Tie bars	Type	Spacing to nearest even foot, in feet	Width	Filler	Dowels
						Inches		
Tennessee	Steel plate	16 g	½-in. round, deformed, 4 ft. long, 5 ft. c. to c.	Expansion	500	2½	Poured or premolded.	None.
Texas	do	18 g	½-in. round, smooth, 4 ft. long, 5 ft. c. to c.	do	200 to 500	3	Poured	None.
Utah	Dummy joint		½-in. round, 4 ft. long, 5 ft. c. to c.	Expansion with dummy joint.[2]	60	¾	Premolded	Do.
Vermont	Construction		⅝-in, round, deformed, 4 ft long, 2 ft 10 ins. c. to c.	Expansion	55	½	Poured or premolded.	Eight ¾-in. round, 2 ft. long, 1 end fixed.
Virginia	None			Construction	Necessary			
Washington	Dummy joint		½-in. round, deformed, 2 ft. long, 4 ft. c to c.	Expansion with dummy joint[6]	40	½ to ⅝	Premolded	Twenty ¾-in. round, 2 ft. long, 1 end fixed. None.
West Virginia	Steel plate Dummy joint or Construction joint.	14 g	None	Expansion with dummy joint.[6]	93	¾	do.	None.
Wisconsin	Steel plate or Dummy joint	18 g	½-in. round, deformed, 4 ft. long, 2 ft. c. to c.	Expansion with dummy joint.[2]	90	1	Poured or premolded	Ten ⅝-in. round, 2 ft. 6 ins. long.

[2] Dummy joints at third points between expansion joints.
[6] Dummy joints halfway between expansion joints.

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*Effect of Salts in Mixing Water on Compressive Strength of Mortar, vol. 8, No. 11, January, 1928.

*Comparative Tests of Crushed Stone and Gravel Concrete in New Jersey, by F. H. Jackson, vol. 8, No. 12, February, 1928. (Supply exhausted.)

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*Determination of Proportions of Constituents in Concrete, by L. G. Carmick, vol. 9, No. 4, June, 1928.

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*The Design of Pavement Concrete by the Water-Cement Ratio Method, by F. H. Jackson, vol. 9, No. 6, August, 1928.

*Field Experiments in the Curing of Concrete Pavements—Report on Cooperative Experiments Conducted by the Maryland State Roads Commission and the U. S. Bureau of Public Roads, by F. H. Jackson and George Werner, vol. 9, No. 7, September, 1928.

*Strength Characteristics of Concrete as Indicated by Core Tests—Results of Compression Tests on Cores Drilled from Maryland Highways Analyzed, by A. N. Johnson, vol. 9, No. 7, September, 1928.

*Modulus of Elasticity of Cores from Concrete Roads—Results of Determinations Made on Cores Drilled from Maryland Highways, by A. N. Johnson, vol. 9, No. 8, October, 1928.

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*Miscellaneous Circular 93 MC. Direct Production Costs of Broken Stone. 25c.

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Effect of Haul on the Cost of Earthwork, by J. L. Harrison, vol. 5, No. 7, September, 1924. (Supply exhausted.)

Cost of Grading with Fresnoes, by J. L. Harrison, vol. 5, No. 8, October, 1924. (Supply exhausted.)

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The Wagon and the Elevating Grader, by J. L. Harrison:
  Part I. An Economic Study of the Wagon-Elevating Grader Combination, vol. 6 .No. 2, April, 1925. (Supply exhausted).
  *Part II. The Influence of Design on Elevating Grader Costs, vol. 6, No. 3, May, 1925.
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  *Part I. Efficient Production, vol. 6, No. 9, November, 1925.
  Part II. Transportation of Materials, vol. 6, No. 10, December, 1925. (Supply exhausted.)
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  *Part IV. Organization and Equipment of a Concrete Paving Operation, vol. 6, No. 12, February, 1926.
  *Part V. Speeding up Construction Work, vol. 7, No. 1, March, 1926.
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*Power-Shovel Operation in Highway Grading, by T. Warren Allen and Andrew P. Anderson.
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  State highways built during year.
  State highways existing at end of year.
  State highway income during year.
  State highway expenditures during year.
  Local roads built during year.
  Local roads existing at end of year.
  Local highway income during year.
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Annual reports. (Free supply.)
  Report of the Chief of the Bureau of Public Roads, 1924.
  Report of the Chief of the Bureau of Public Roads, 1925.
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*Department Bulletin 136 D, Highway Bonds, 20c.
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Department Bulletin 1279 D, Rural highway mileage, income and expenditures 1921 and 1922. 15c. (Free supply available.)
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*Road Bond Issues in Relation to Total Debt, by Henry R. Trumbower, vol. 5, No. 3, May, 1924. (Supply exhausted.)
*The Incidence of the Highway Tax Burden, by Henry R. Trumbower, vol. 5, No. 4, June, 1924. (Supply exhausted.)
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*Motor Vehicle Fees and Gasoline Taxes, by Henry R. Trumbower, vol. 5, No. 7, September, 1924. (Supply exhausted.)
*Constitutionality of Motor Vehicle License Fees and the Gasoline Tax by Henry R. Trumbower, vol. 5, No. 9, November, 1924. (Supply exhausted.)
*Highway Income from the Motor Vehicle, by Henry R. Trumbower, vol. 5, No. 11, January, 1925. (Supply exhausted.)
*Common Carrier Truck Fees and Taxes, by Henry R. Trumbower, vol. 6, No. 11, January, 1926. (Supply exhausted.)
*Urban Aspects of Highway Finance, by Jacob Viner, vol. 6, No. 11 (supply exhausted), and No. 12, January and February, 1926.
*Collection and Disposition of Motor Vehicle Revenues, by Henry R. Trumbower, vol. 7, No. 11, January, 1927. (Supply exhausted.)
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*North Carolina County Road and Finance Survey. Report of a Cooperative Investigation by the North Carolina State Highway Commission, the North Carolina State Tax Commission, and the U. S. Bureau of Public Roads, vol. 11, No. 12, February, 1931.
*Toll Roads, by H. H. Kelly, vol. 12, No. 1, March, 1931.

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Report of a Survey of Transportation on the State Highways of Vermont, 1927.
Report of a Survey of Transportation on the State Highways of New Hampshire, 1927.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio, 1928.
Report of a Survey of Transportation on the State Highways of Pennsylvania, 1928.
Report of a Survey of Traffic on the Federal-aid Highway Systems of Eleven Western States, 1930.

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*Transportation of Milk by Motor Truck, by H. R. Trumbower, vol. 5, No. 5, July, 1924.
*Traffic Control and Safety, by E. W. James, vol. 5, No. 6, August, 1924. (Supply exhausted.)
*Transportation of Hogs by Motor Truck, by E. L. Browne, vol. 5, No. 6, August, 1924. (Supply exhausted.)
*A Study of Motor Vehicle Accidents in Montana, Oregon, and Washington, by A. C. Rose, vol. 5, No. 12, February, 1925.
*Transverse Distribution of Motor Vehicle Traffic on Paved Highways, by J. T. Pauls, vol. 6, No. 1, March, 1925.
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*Commodity Transportation by Motor Truck, by J. G. McKay, vol. 6, No. 6, August, 1925. (Supply exhausted.)
*Colors and Forms of Traffic Signals, vol. 6, No. 6, August, 1925. (Supply exhausted.)
*Railroad Abandonments and their Relation to Highway Transportation, by H. R. Trumbower, vol. 6, No. 8, October, 1925.
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*Truck is a Big Factor in Fruit Transport, vol. 9, No. 6, August, 1928.
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Department Bulletin 1486 D, Highway Bridge Location. 15c. (Free supply available.)

Technical Bulletin 55 T, Highway Bridge Surveys. 20c. (Free supply available.)

Technical Bulletin 265 T, Electrical equipment on Movable Bridges. 35c. (Free supply available.)

Reprints from Journal of Agricultural Research. (Free supply available):

- Tests of Three Large-Sized Reinforced-Concrete Slabs under Concentrated Loading. Vol. 6, No. 6, D-8.
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*Earth Pressures Against Abutment Walls Measured with Soil Pressure Cells, by J. V. McNary, Vol. 6, No. 5, July, 1925.

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*Concrete Compared with Timber for Highway Bridge Floors, by O. L. Grover, Vol. 7, No. 8, October, 1926.

*Analysis of Concrete Arches, by W. P. Linton and C. D. Geisler. Part I, vol. 8, No. 4, June, 1927; Part II, vol. 8, No. 5, July, 1927. Available as a reprint from Public Roads under a single cover, 10c.

*Tests of the Delaware River Bridge Floor Slabs, by George W. Davis, vol. 8, No. 8, October, 1927.

*Foundation Pile-Head Bond and Anchorage Tests, by George W. Davis, vol. 9, No. 9, November, 1928

*Loading Tests on a Reinforced Concrete Arch—Report on Tests Made on Yadkin River Bridge in North Carolina, by Albin L. Gemeny and W. F. Hunter, vol. 9, No. 10, December, 1928.

*Model Analysis of a Reinforced Concrete Arch—Report on a Cooperative Study by the Johns Hopkins University and the United States Bureau of Public Roads in Connection with Yadkin River Bridge Tests, by J. T. Thompson, vol. 9, No. 11, January, 1929.

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*General Formula for Waterways, by C. S. Jarvis, vol. 6, No. 12, February, 1926.

*Retards in Stream Control, by John R. Chamberlain, vol. 7, No. 3, May 1926. (Supply exhausted.)

*Maximum Stream Flow—Formula for General Use, by C. E. Grunsky, vol. 7, No. 4, June 1926. (Supply exhausted.)

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*Some Aspects of Flow of Water Around Bends and Bridge Piers, by D. L. Yarnell, vol. 10, No. 2, April, 1929.

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*Department Bulletin 347 C. Methods for the Determination of the Physical Properties of Road-Building Rock, 10c.

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*Design of a Constant Temperature Moist Closet, by Wallace F. Purrington, vol. 7, No. 12, February, 1927.

*Relation between sodium sulphate soundness test and absorption of sedimentary rock, by D. O. Woolf, vol. 8, No. 10, December, 1927.

*Cantilever testing apparatus for mortar beams, by D. O. Woolf, vol. 9, No. 3, May, 1928.

*New moist closet and storage tank apparatus, by D. O. Woolf, vol. 9, No. 4, June, 1928.

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*How Massachusetts is Improving Her Roadsides, by R. E. Tribou, vol. 9, No. 2, April, 1928.

*Parkway features of interest to the highway engineer, by E. W. James, vol. 10, No. 2, April, 1929.

*Roadside plan and progress in Massachusetts, by James H. Taylor, vol. 10, No. 6, August, 1929.

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Miscellaneous Publication 76 M. The results of Physical Tests of Road-Building Rock, 25c. (Free supply available.)

*Department Bulletin 583 D. Report on Experimental Convict Road Camp, Fulton County, Georgia, 25c.

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*Cooperative Survey of Corrugated Metal Culverts on the Austin-San Antonio Post Road, by E. F. Kelley, vol. 11, No. 9, November, 1930.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

APRIL 30, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$				$	$				$	
Alabama	2,379.6	2,008,496.70	982,352.29	88.5		88.5						5,469,694.24	Alabama
Arizona	1,250.7	2,010,847.26	1,438,177.82	122.4	1.2	123.6	396,558.93	298,569.20	3.7	21.2	24.9	1,204,333.27	Arizona
Arkansas	1,955.0	2,686,026.78	1,243,958.18	64.8	50.2	115.0	362,032.53	181,016.26	17.3		17.3	2,090,050.06	Arkansas
California	2,301.2	9,724,527.57	4,436,914.36	205.1	38.7	243.8	3,445,989.62	1,623,236.20	68.8	17.0	85.8	206,985.78	California
Colorado	1,575.4	4,388,534.62	2,319,304.94	185.5	50.3	235.8	346,846.72	168,815.57	14.2	.5	14.7	2,077,729.82	Colorado
Connecticut	288.5	3,084,244.77	1,151,757.90	22.9		22.9	227,508.93	104,160.72	4.0		4.0	610,083.83	Connecticut
Delaware	345.2	760,393.25	380,196.62	30.5		30.5	210,556.00	105,278.00	15.9		15.9	167,140.82	Delaware
Florida	617.5	4,232,614.05	1,935,392.29	111.3		111.3	241,813.52	120,906.75	12.2		12.2	2,880,867.20	Florida
Georgia	3,100.0	5,385,405.11	2,480,059.86	106.1	151.0	257.1	1,827,452.26	883,262.82	44.1	45.0	89.1	1,306,477.79	Georgia
Idaho	1,485.0	1,888,362.34	1,097,031.63	90.0	80.7	170.7	500,986.83	302,407.99	34.4	38.5	72.9	1,182,677.78	Idaho
Illinois	2,632.6	24,676,208.29	11,511,846.55	736.4	28.7	765.1	3,481,472.92	1,389,059.28	107.4	12.8	120.2	2,077,789.15	Illinois
Indiana	1,788.6	9,343,751.67	4,592,634.05	278.4	3.2	281.6	3,221,784.39	1,572,088.33	133.7	4.4	138.1	603,704.86	Indiana
Iowa	3,390.8						4,095,531.28	1,912,725.18	191.2	54.0	245.2	857,080.02	Iowa
Kansas	3,656.8	4,723,234.79	2,279,133.06	280.0	15.5	295.5	2,682,780.55	1,310,276.73	125.1	219.1	344.2	1,017,871.44	Kansas
Kentucky	1,869.8	2,584,916.67	1,148,079.52	140.9	4.0	144.9	879,379.99	390,003.50	38.5	46.0	84.5	1,504,550.07	Kentucky
Louisiana	1,557.0	6,794,954.68	3,173,425.78	96.2	10.6	106.8	956,563.32	478,281.66	.5		.5	815,611.31	Louisiana
Maine	706.0	1,804,346.15	789,080.76	31.4		31.4	2,236,243.22	939,451.37	75.5		75.5	421,951.71	Maine
Maryland	776.8	338,200.10	97,983.36	8.9		8.9	645,409.70	297,634.62	26.7	.6	27.3	588,690.84	Maryland
Massachusetts	814.8	7,521,359.24	3,084,570.72	66.8	.1	66.9	101,788.80	50,894.40	3.2		3.2	637,533.67	Massachusetts
Michigan	1,998.3	9,651,288.88	4,486,294.53	425.7	50.9	476.6	282,361.19	56,765.37	13.1	6.1	19.2	3,139,277.43	Michigan
Minnesota	4,144.8	3,654,812.07	1,250,809.01	65.7	103.1	168.8	8,335,406.65	2,689,063.46	154.8	210.1	364.9	35,903.63	Minnesota
Mississippi	1,805.1	3,843,939.79	1,888,599.47	162.2	75.9	238.1	63,967.53	31,983.76	6.4		6.4	5,324,677.74	Mississippi
Missouri	2,976.7	3,175,745.17	1,356,569.71	96.3		96.3	5,801,737.71	2,532,888.42	143.0	100.2	243.2	586,603.68	Missouri
Montana	2,717.8	4,031,970.32	2,268,927.77	351.7	17.6	369.3	399,015.07	223,670.27	57.2	.5	57.7	3,670,527.48	Montana
Nebraska	4,172.8	4,269,855.99	2,101,892.43	190.2	66.3	256.5	290,799.88	145,301.24	13.3		13.3	2,638,132.15	Nebraska
Nevada	1,327.2	1,250,055.74	968,855.55	38.7	100.0	138.7	786,529.05	698,603.39	3.3	70.1	73.4	614,139.07	Nevada
New Hampshire	418.4	636,114.46	246,912.44	12.1	2.5	14.6						553,268.42	New Hampshire
New Jersey	605.7	3,760,915.32	1,327,092.02	36.3	.3	36.6						1,592,693.57	New Jersey
New Mexico	2,196.9	2,338,839.50	1,516,435.95	115.9	17.7	133.6	1,016,475.46	647,378.40	41.1	6.5	47.6	876,109.05	New Mexico
New York	3,262.5	11,445,100.00	5,288,075.00	267.7		267.7	7,862,149.00	3,031,744.50	164.2		164.2	2,752,586.03	New York
North Carolina	2,222.8	1,311,744.48	632,975.21	68.9		68.9	232,703.74	116,351.85	11.1	9.0	20.1	3,801,553.52	North Carolina
North Dakota	5,076.5	2,533,360.10	1,271,292.64	271.1	210.7	481.8	1,751,459.58	889,107.72	148.1	337.8	485.9	1,585,594.75	North Dakota
Ohio	2,854.4	6,347,538.56	2,227,113.68	91.5	18.4	109.9	3,643,238.52	1,278,440.74	79.8	17.6	97.4	3,004,724.56	Ohio
Oklahoma	2,255.3	4,053,069.67	1,921,207.88	113.2	61.2	174.4	1,243,378.68	629,434.87	112.1	6.8	118.9	1,737,715.26	Oklahoma
Oregon	1,529.0	4,080,678.53	2,262,172.42	119.3	47.1	166.4	779,316.22	467,080.91	5.2	40.0	45.2	1,321,125.41	Oregon
Pennsylvania	3,020.0	1,722,045.11	681,721.34	38.0		38.0	5,176,651.13	2,232,360.32	158.6		158.6	2,943,259.13	Pennsylvania
Rhode Island	255.4	890,586.30	441,180.98	16.6	2.2	18.8						365,211.86	Rhode Island
South Carolina	2,008.6	2,477,211.20	1,054,161.02	56.9	37.4	94.3	804,446.34	386,000.00	25.6	13.3	38.9	1,042,551.03	South Carolina
South Dakota	4,038.8	3,451,363.76	1,951,959.42	230.1	181.9	412.0	269,938.45	187,615.53	49.1		49.1	1,080,793.57	South Dakota
Tennessee	1,669.0	1,018,377.27	498,947.20	33.6	10.7	44.3	574,928.81	279,752.44	26.4		26.4	3,545,556.85	Tennessee
Texas	7,630.8	14,372,646.52	6,531,027.59	776.7	187.2	963.9	3,000,299.08	1,334,024.90	136.7	108.8	245.5	5,121,710.29	Texas
Utah	1,212.3	908,505.04	646,990.90	70.5	6.6	77.1	723,417.96	535,329.16	32.8	85.9	118.7	869,349.93	Utah
Vermont	339.1	104,832.58	52,345.06	4.3		4.3	1,086,289.88	492,855.54	31.8		31.8	89,110.73	Vermont
Virginia	1,917.1	1,985,427.19	992,579.58	117.2		117.2	159,197.59	79,598.79	5.4		5.4	2,065,478.19	Virginia
Washington	1,184.3	2,755,425.96	1,321,468.56	84.7	11.1	95.8	820,567.76	401,300.00	31.3	10.7	42.0	1,225,142.23	Washington
West Virginia	882.9	1,882,429.90	835,314.59	44.7	10.5	55.2	473,910.17	213,288.81	10.9		10.9	1,038,154.78	West Virginia
Wisconsin	2,672.7	3,874,672.71	1,572,633.84	131.0	7.8	138.8	1,796,381.33	786,000.00	42.0	24.7	66.7	1,518,163.91	Wisconsin
Wyoming	1,956.3	3,240,861.84	1,946,522.75	225.1	136.4	361.5	334,624.92	214,182.54	16.5	32.1	48.6	438,609.36	Wyoming
Hawaii	74.2	700,101.47	312,800.22	18.9		18.9	424,052.28	349,164.42	6.7	3.6	10.3	1,691,776.18	Hawaii
TOTALS	100,917.0	199,725,939.47	93,996,778.45	6,940.9	1,797.7	8,738.6	73,994,243.99	33,057,355.93	2,442.9	1,542.9	3,985.8	82,020,323.45	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U S GOVERNMENT PRINTING OFFICE 1932

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 4 JUNE, 1932

SECTION OF MOUNT VERNON MEMORIAL HIGHWAY ON COLUMBIA ISLAND, D. C.

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 4 June, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

STABILIZATION BY DRAINAGE OF MUCK AND SAND FILL

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer Division of Tests, United States Bureau of Public Roads

COLUMBIA ISLAND, on which a section of the Mount Vernon Memorial Highway, 4,000 feet in length, has been constructed, is a body of land which was built up from the waste material resulting from the hydraulic dredging operations performed in deepening the Potomac River Channel between the Highway Bridge and Georgetown, D. C. In many places the material pumped in was extremely unstable and, in the condition in which it was encountered on the island, was unfit for use as a highway subgrade. The measures employed in improving the stability of these portions of the grade which were unsatisfactory furnish excellent examples of some of the fundamental principles of highway drainage and illustrate the behavior of a class of soil frequently encountered in highway construction.

At the time it was desired to start grading operations on Columbia Island a section about 300 feet in length between stations 10+00 and 13+00 (see fig. 1) was covered with water and was so unstable that a person attempting to walk along the center line would sink in to his knees.

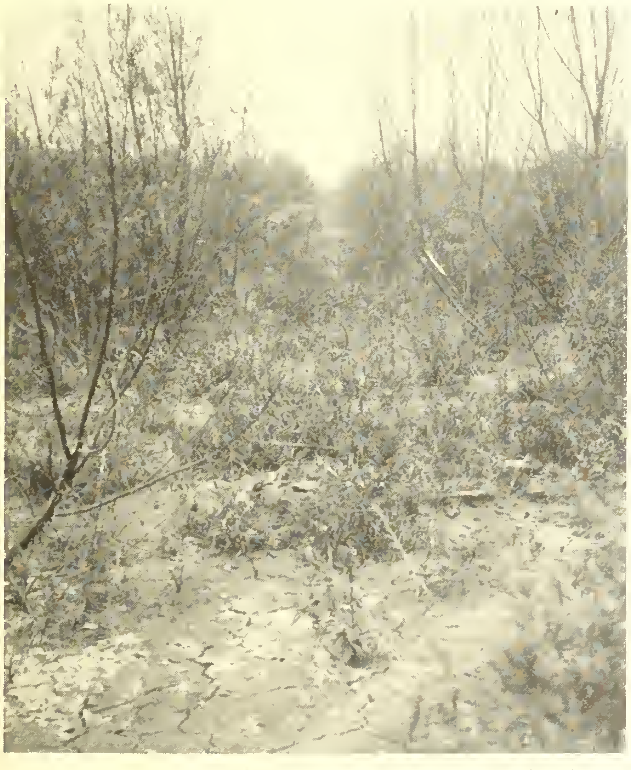

FIGURE 1.—LOOKING NORTH FROM STATION 13+00, ALONG THE CENTER LINE, BEFORE DRAINAGE

The remainder of the grade, between stations 13+00 and 50+00, was perfectly dry on the surface and was firm underfoot. It was, consequently, assumed to be in a stable condition and equipment was moved in. This step revealed the presence of soft undersoil in many places.

The power shovel was bogged down in several instances (see fig. 2) even though it was traveling over timber mats and the trucks soon broke through the harder surface crust. It was evident that stabilization would have to be effected before construction could be attempted.

PERMEABLE SAND AND IMPERMEABLE MUCK COMPOSE SOIL OF ISLAND

Preliminary observations revealed that, in general, the island consists of only two distinct types of soil. One is a clean, permeable river sand. The other is a silty river bottom muck, almost impermeable in a compact state but having relatively high shrinkage properties which, under the action of evaporation, produced large cracks and fissures which would act as water carriers. The character of this muck is shown in Figure 3. Mixtures of the two soils were, of course, encountered.

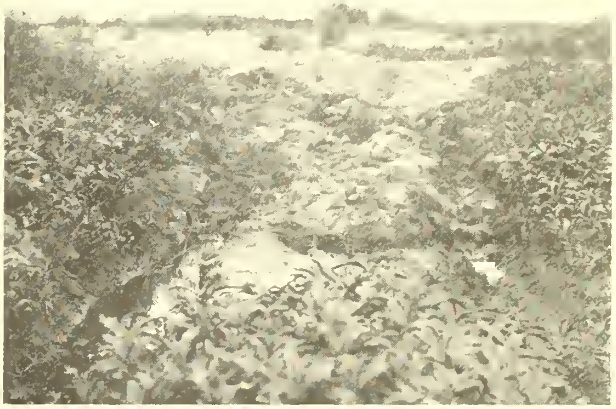

FIGURE 2.—WET AREA AT STATION 20+00. POWER SHOVEL WAS STUCK AT THIS POINT

Table 1 gives the mechanical analysis and the subgrade soil constants of each of the two soils. The nature of these test constants and the methods of obtaining them are discussed in a paper entitled, "Subgrade Soil Constants, their Significance, and their Application in Practice," Part I, PUBLIC ROADS, vol. 12, No. 4, June, 1931; and Parts II and III, PUBLIC ROADS, vol. 12, No. 5, July, 1931. The classification of subgrade soils according to their characteristics into groups A–1 to A–8 is also discussed in these articles.

The test results shown in Table 1 indicate that the sand found on Columbia Island has the properties of the A–3 subgrade group. The muck is an A–8 subgrade. The significance of these groupings may be understood from the following quotations:[1]

> *Group A–3.*—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.
>
> *Group A–8.*—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

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[1] PUBLIC ROADS, vol. 12, No. 4, June, 1931, p. 105.

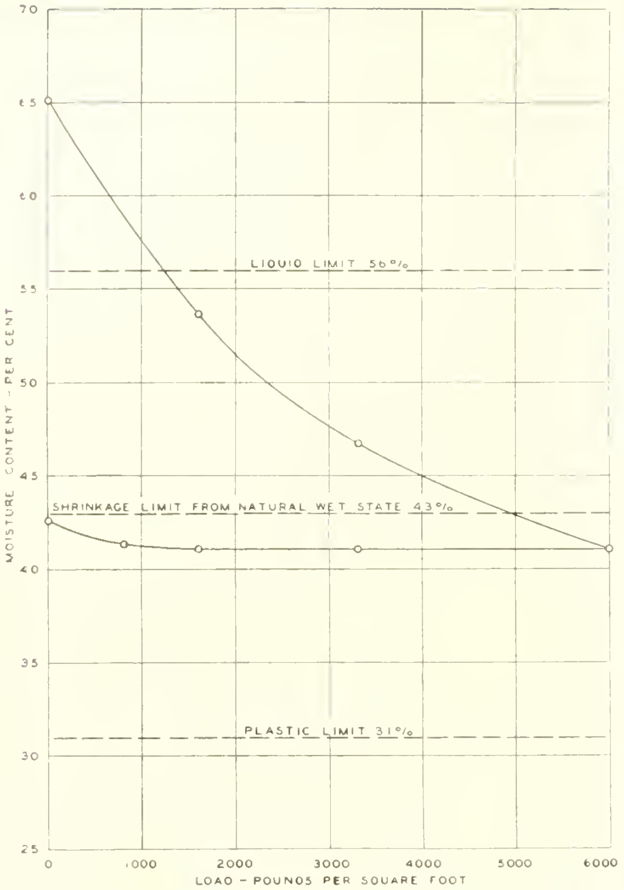


Figure 5.—Compression Test of Muck Soil in Disturbed State

These test data reveal information on two important points. On one hand they show that in order to stabilize a layer of the soil of indefinite thickness by drainage, its moisture content must be reduced below 31 per cent, and furthermore, that it would require a pressure much greater than 6,000 pounds per square foot actually to squeeze the water out. On the other hand, they show that if this muck serves as a base for a new top layer of soil it will support 1,000 pounds per square foot, or an 8-foot thickness of gravel, even with 57 per cent of moisture; and that with a moisture content of 61 per cent it will support a 4-foot layer of ground.

If only the character of the material were known, and not the conditions under which it exists in place, a comparison of the results likely to be furnished by drainage and by covering would suggest covering to be the more logical procedure in uniform deposits of great depth where the ground water table is high. In stratified deposits, however, the removal by drainage of water which is the source of detrimental capillarity may serve to very quickly stabilize the surface muck layer. Columbia Island, according to a subgrade survey, was found to be an excellent representation of the type of stratified soil profile in which such stabilization could be accomplished quickly and economically.

SOIL SURVEY INDICATED CHARACTER OF DEPOSITS AND DISTRIBUTION OF MOISTURE

In connection with this survey borings made every 50 feet, along and on both sides of the center line where trouble was encountered, brought to light several interesting conditions not usually found in natural soil deposits. The soil profile is shown in Figure 6. The sand and muck occurred in layers and pockets varying from an inch to several feet in thickness. In places wet sand occurred above dry muck; in others, dry sand over saturated muck; in others, wet muck over wet sand; and in still others, dry muck over wet sand.

Finally, in several instances perched water tables were encountered. All of the free water surfaces were above the elevation of the river; but in certain places free water occurred in several layers vertically above each other with relatively dry material in between.

The comparatively small pockets of saturated sand and muck presented no particular difficulties. The prevailing condition, particularly in the area between stations 10+00 and 13+00, where the soil was the least stable, was that of a layer of saturated muck about 6 feet thick underlain by a bed of saturated sand.

A consideration of the characteristics of this material furnished important information. Those test results which showed how the fill would shrink if the moisture content were to be reduced from 115 to 31 per cent, also showed that if this soil had a chance to dry out by evaporation it would crack to such an extent that it would transmit water freely. It could be readily seen that if the water were taken away from the bottom of the muck, evaporation from the top would dry out the soil. These two factors pointed in only one direction and that was to drain out the sand layer and let the characteristic properties of the muck do the rest.

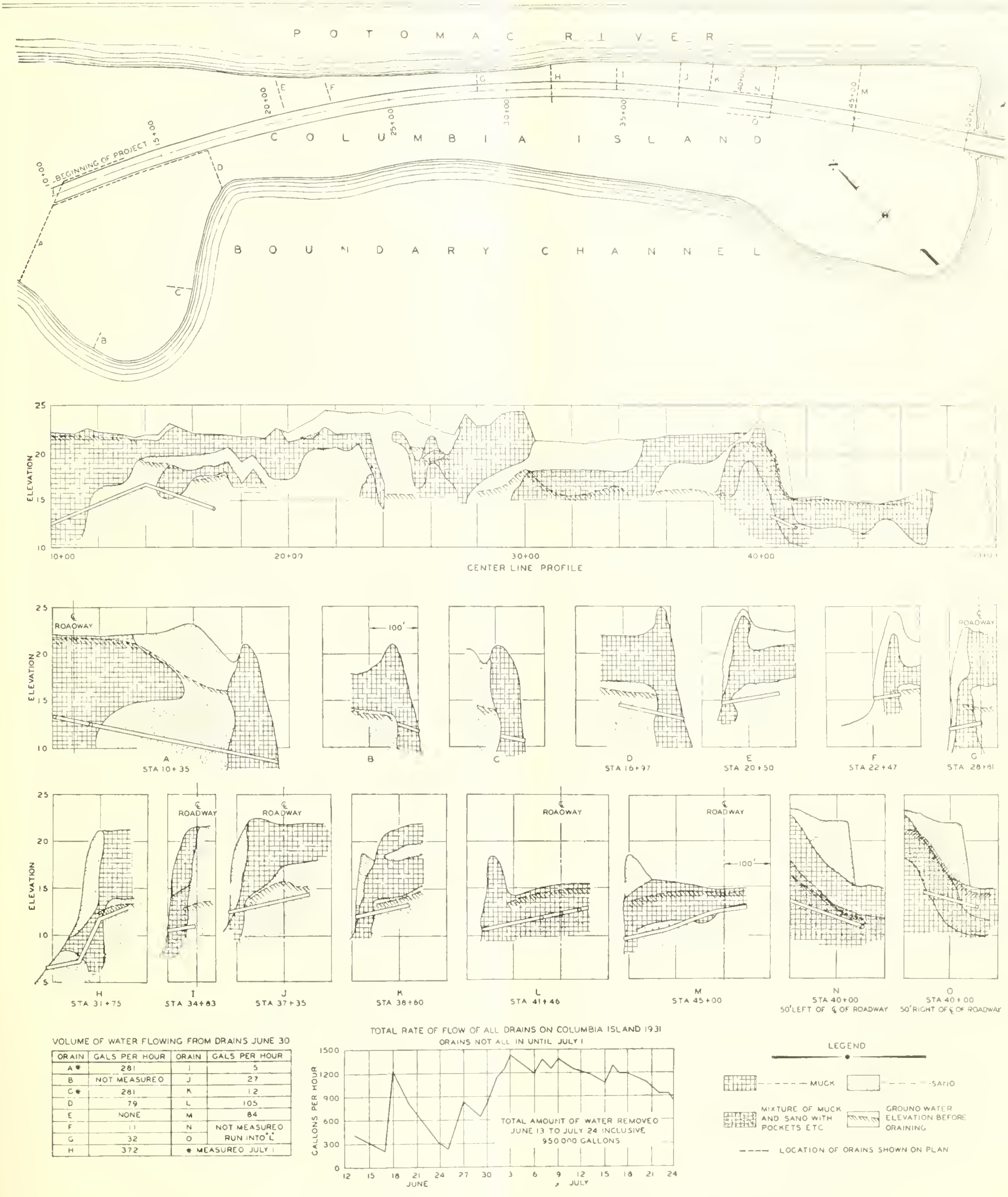
Knowledge of the manner in which the island was formed was very helpful in selecting the location and depth at which the drains should be placed. The island was built up over a period of years from the material dredged out of the river channel. It was thus constructed in a series of "lifts."

Each lift was formed by building a bank or dike around an area and pumping in sand and muck from the river bottom. The area inclosed by the dike formed a settling basin and the excess water was allowed to escape over a timber spillway.

Great pains were taken to make the dike impervious as this reduced the danger of washouts. Wherever seepage through the banks was noticed, soft muck was spread over the inner faces of the dikes with hoes until the dikes were effectively sealed. In this manner impermeable walls were built up around the whole island.

The coarse material naturally settled out of suspension faster than the fine material and was deposited nearer to the end of the discharge pipe. This action had the effect of separating the coarse from the fine material and several instances were encountered where a large amount of sand was surrounded by impermeable muck forming a reservoir.

Since the island was formed in successive lifts with a considerable time interval between the placement of each lift, there was time for each one to become more or less consolidated and covered with vegetable growth. Consequently, a layer of slightly decomposed organic matter was often encountered and a difference in permeability was noticed between successive lifts even where the material appeared to be the same. Furthermore, the material from which the different lifts were formed was often obtained from different parts of the river, with the result that successive layers often consisted of widely different materials.



VOLUME OF WATER FLOWING FROM DRAINS JUNE 30

DRAIN	GALS PER HOUR	DRAIN	GALS PER HOUR
A *	281	I	5
B	NOT MEASURED	J	27
C *	281	K	12
D	79	L	105
E	NONE	M	84
F	11	N	NOT MEASURED RUN INTO "L"
G	32	O	
H	372	* MEASURED JULY 1	

Figure 6.—Map and Soil Profile of Columbia Island

PLAN OF DRAINAGE DESCRIBED

In general, the drainage plan consisted of three essential operations, namely: (1) Removal of free surface water, (2) removal of brush and vegetation, and (3) tapping of the impermeable dike and placing drains in such a manner as to remove all underground water which was trapped in layers or pockets.

The surface water was removed from the wet area between stations 10+00 and 13+00 in an attempt to dry it up enough to make it possible to work in this area. Shallow surface ditches (figs. 7, A and 8, A) were dug from the dikes back toward the center line. A great quantity of water was carried off in this manner and as soon as the water on the surface was removed

THE SEGREGATION OF WATER IN CONCRETE PLACED IN DEEP FORMS

By F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer, United States Bureau of Public Roads

IT IS well known that during the continuous placing of concrete in deep forms such as columns, piers, abutments, retaining walls, etc., there is a tendency for water within the concrete to rise as the form is filled. This results in an accumulation of excess water in that portion of the concrete at and near the upper surface of each continuously placed section with the consequent formation of layers of relatively weak, porous concrete at the fill planes. It is a matter of common observation that such concrete, when exposed to the weather, deteriorates much more rapidly than the adjacent concrete at the bottom of the next lift. Typical examples of this action are shown in Figure 1. It has been generally assumed that this difference in the rate of weathering is due in large part to excessive water, generally known as water gain, although little or no data have been published which show quantitively either the actual effect of water gain on strength and durability or the relative influence of various factors such as grading of aggregates, consistency of concrete, area of section, method of placing, tamping, etc., on the extent to which water gain takes place.

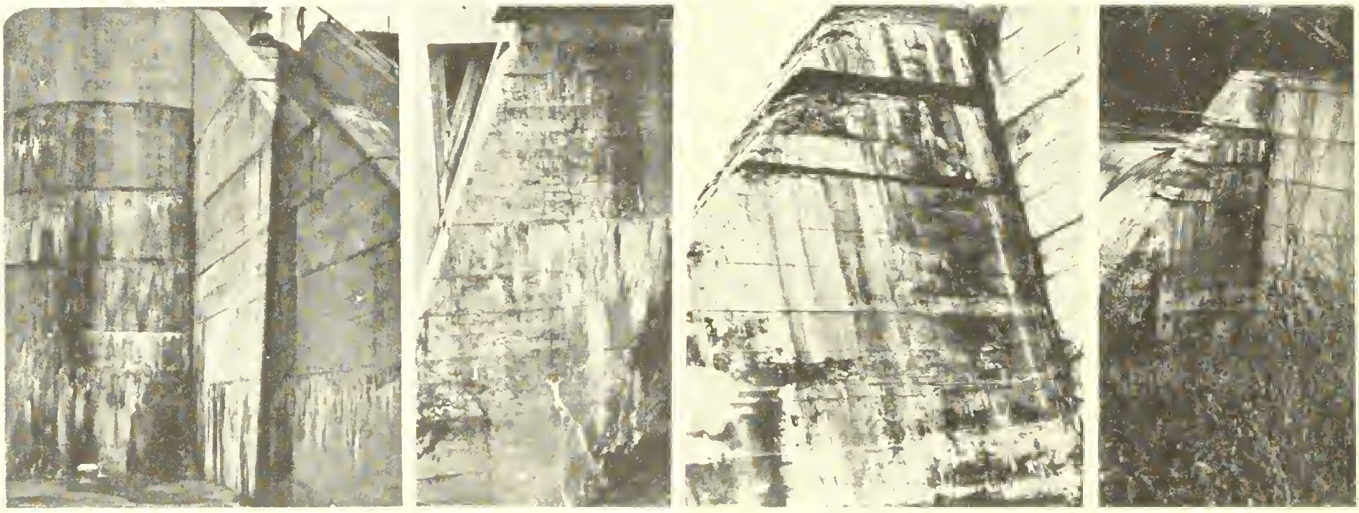


Figure 1.—Typical Examples of Disintegration at Fill Planes

The problem appeared to the bureau to be of sufficient interest to justify a preliminary series of tests under certain job conditions to determine the extent to which water gain developed and its effect upon the quality and uniformity of the concrete.

It was not possible to cover a wide range in test conditions; and the applicability of the data is restricted to the conditions under which it was obtained. While forms of different heights were used, they were all of one width, 8 inches. Methods of placing included both spading and vibrating. The fine aggregate used was a relatively fine sand. The consistency of the concrete was varied from 2 to 8 inches.

DESCRIPTION OF TESTING PROCEDURE

The tests were made by investigating the variations in composition and strength of concrete in a series of 8 by 48 inch vertical wall columns cast as units and ranging in height from 3 to 12 feet. Four different heights of column were used, 3 feet, 6 feet, 9 feet, and 12 feet. One side of each form was provided with a series of ports equipped with closely fitting doors, spaced about 2 feet apart vertically. From 1 to 1½ hours after placing, depending upon consistency, a sufficient quantity of the fresh concrete was obtained from each of these ports to permit the fabrication of one 6 by 12 inch cylinder as well as an analysis of the fresh concrete. A diagrammatic view of one set of columns is shown in Figure 2. This view shows the location of the ports (indicated by circles) and also the location from which cores were subsequently drilled from the hardened concrete (indicated by crosses). A ⅞-inch vertical dividing strip divided each column into two 8 by 24 inch sections, as indicated. This division was made in order to permit the withdrawal of samples of wet concrete without disturbing the concrete from which the cores were to be drilled.

A total of 33 columns were cast. With the exception of series K, which consisted of one 12-foot column only, they were constructed in sets of four, one of each of the four heights being cast on the same day. Various consistencies were employed, ranging from 2 to 8 inch slump, as well as two methods of consolidating, hand spading and vibrating with an electric vibrator attached to the form. Table 1 gives the consistency and method of compacting employed in each series of four columns.

Table 1.—*Consistency and method of compaction used in each series*

Series	Average slump in inches	Method of compaction	Series	Average slump in inches	Method of compaction
A	4	Spaded.	F	8	Spaded.
B	8	Do.	G	5	Vibrated.
C	4	Vibrated.	H	2	Do.[1]
D	3	Do.	K [2]	(3)	Do.
E	4	Spaded.			

[1] Sides and ends of forms also spaded.
[2] 12-foot column only cast.
[3] Slump of concrete in the lower 1 foot, 2 inches. Slump of concrete in remaining 11 feet, 4 inches.

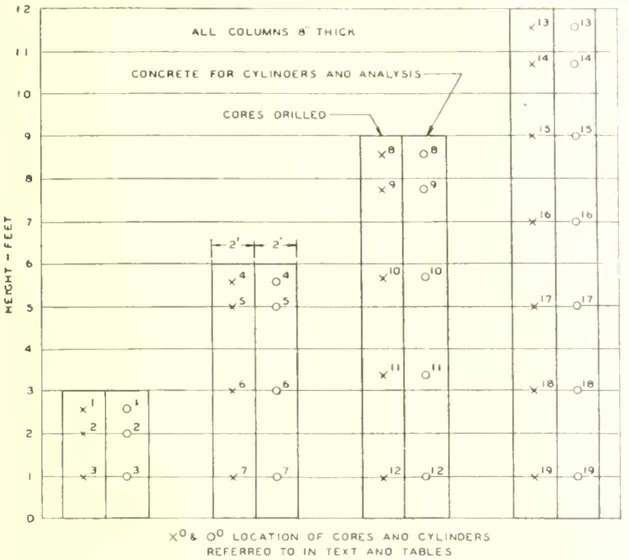

ALL COLUMNS 8" THICK
CONCRETE FOR CYLINDERS AND ANALYSIS
CORES DRILLED
HEIGHT - FEET
2'
2'
0
1
2
3
4
5
6
7
8
9
10
11
12
x1 o1 x2 o2 x3 o3
x4 o4 x5 o5 x6 o6 x7 o7
x8 o8 x9 o9 x10 o10 x11 o11 x12 o12
x13 o13 x14 o14 x15 o15 x16 o16 x17 o17 x18 o18 x19 o19
x⁰ & o⁰ LOCATION OF CORES AND CYLINDERS REFERRED TO IN TEXT AND TABLES

Figure 2.—Location of Concrete Specimens Taken From Columns

A 1:2:4 dry-rodded volume mix, the aggregates being Potomac River sand and gravel, was used throughout the series. The results of tests on the cement are given in Table 2 and the results of tests on the aggregates in Tables 3 and 4. It will be noted that a relatively fine sand having a fineness modulus of 2.49 was employed. This is somewhat finer than the concrete sands employed in general in the Northern States, but is similar to the grading of many natural sands in other parts of the country.

Table 2.—*Tests of cement*

Fineness, percentage retained on 200-mesh sieve	7.8
Time of set (Gillmore)—	
Initial	2 hours 35 minutes
Final	6 hours 00 minutes
Soundness	Satisfactory
Normal consistency, per cent	24

TENSILE STRENGTH

[Pounds per square inch, 1:3 Ottawa sand mortar briquets]

	1 day	3 days	7 days	28 days
	180	290	345	425
	170	315	430	420
	155	305	370	425
Average	170	305	380	425

The concrete was mixed one minute in a 1-sack mixer. Water was added at the mixer until the required slump was obtained and this amount plus the free water in the aggregates was taken as the total from which the water-cement ratios at time of mixing (shown in fig. 11) were computed.

The forms were placed on a grade so that the top of the 12-foot forms was only about 5 feet above the top of the 3-foot forms A scaffold was built from the mixer to the forms and the concrete hauled in buggies and dumped directly into the forms, each buggy depositing a layer of concrete about 9 inches in depth. The buggies were dumped in such a way that the concrete from the left side of the buggy went into that section of the form from which concrete for analysis and cylinders were taken, while that from the right side of the buggy went into that section from which the cores were drilled. By this method of manipulation any segregation in the buggies was divided between the two sections of the forms. Cylinders were made from concrete taken from the mixer as the columns were being cast. In Figure 3 a general view of the project is shown.

Table 3.—*Tests of fine aggregate*

Sieve analysis:	
Total retained on—	Per cent
No. 4 sieve	1
No. 8 sieve	10
No. 16 sieve	20
No. 30 sieve	40
No. 50 sieve	83
No. 100 sieve	95
Fineness modulus	2.49
Silt and clay, per cent	2.8
Apparent specific gravity	2.65
Weight per cubic foot (dry-rodded), pounds	100

Table 4—*Tests of coarse aggregate*

Sieve analysis:	
Total retained on—	Per cent
1¼ inch sieve	0
¾-inch sieve	5
⅜-inch sieve	65
No. 4 sieve	97
Fineness modulus	6.67
Apparent specific gravity	2.55
Weight per cubic foot (dry-rodded), pounds	102
Percentage absorption—	
30 minutes	.85
24 hours	.91

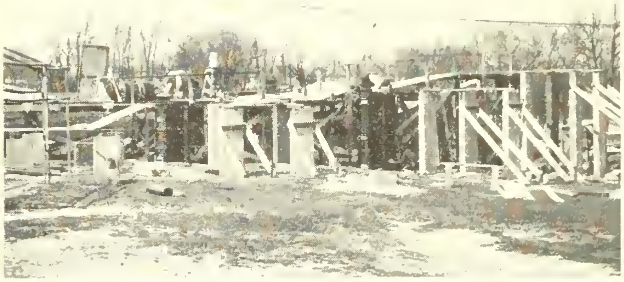


Figure 3.—General View Showing Hardened Columns of Different Heights, and Forms Being Reset

There were two methods used for compacting the concrete, spading and vibrating. The spading method was as follows: After each buggy was dumped the sides and ends of the forms were spaded with a sidewalk scraper with 1-inch holes punched in it to permit the fine material to come in contact with the form. The concrete was then cut with a vertical movement of the spading tool 18 times, 6 times in the center, and 6 times each in two places about midway between the center and the ends. Two men spaded simultaneously, one man spading the right section while the other man spaded the left. (See fig. 4.) In all cases the spading tools were driven down into the previous layer of concrete. This procedure was continued until the form was filled, after which the top was struck off and wet burlap applied. The method of vibrating was as follows: The electric vibrators were attached to the forms by clamping them to the 2 by 4 inch battens as shown in Figure 5. Each vibrator consisted essentially of an electric motor carrying an eccentric weight on the shaft and operating at a frequency of 3,600 vibrations per minute. On the 3-foot forms one vibrator, clamped in one position near the top, was used. On the 6-foot

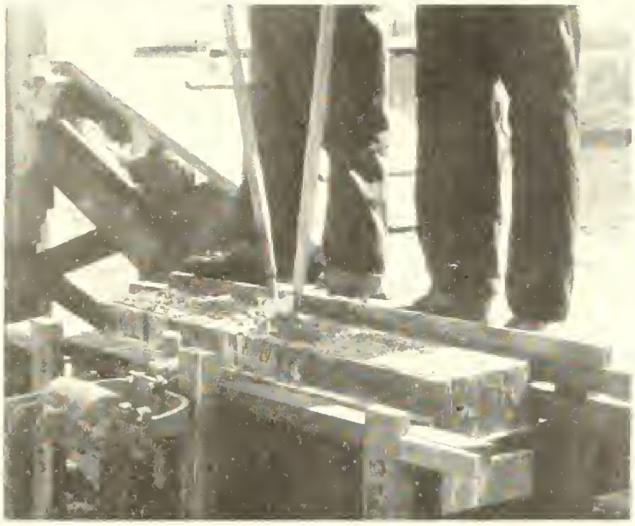


Figure 4.—Showing Method of Simultaneous Spading and Vibrating

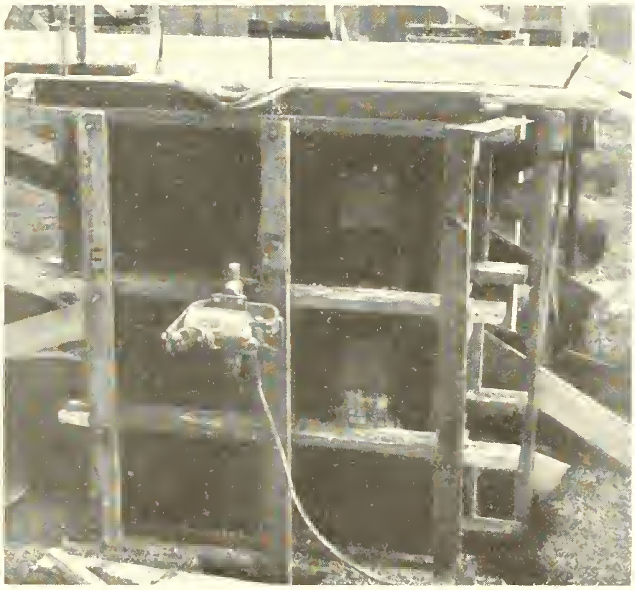


Figure 5.—Vibrator Clamped to Batten of 6-Foot Form

forms one vibrator was used but was moved from near the bottom to a position about two-thirds up, as the form was filled. On the 9-foot forms two vibrators, placed on opposite sides, were used. They were staggered as to height and the lower vibrator was moved up to a position near the top as the forms were filled. On the 12-foot forms (for series C and D), two vibrators were used in a manner similar to that used on the 9-foot forms. In series G, H, and K, both vibrators were placed on the same side. This was done in order to permit the installation of pressure cells for measuring the pressure exerted by the wet concrete. The results of tests for distribution of pressure was published in Public Roads, vol. 12, No. 1, March, 1931. In series C the vibrators were run only a sufficient length of time (5 to 20 seconds) after each buggy was dumped to level off the concrete. In the other vibrated series the vibrators were run continuously while the forms were being filled. In series H the forms were spaded on the sides and ends in addition to being vibrated. This was done in order to eliminate as far as possible the pitted faces of the vibrated columns. Figure 6 shows this condition. The column shown in the center background was vibrated while those in the right and left foreground were spaded. The tests indicated that spading the sides of the forms during vibration materially reduced the amount of surface pitting. In all cases wet burlap was applied as soon as the forms were filled as in the spaded series.

It was found necessary to brace and tie the form very securely when using the vibrators in order to avoid springing the forms.

Series K (12-foot column only) was placed in order to study the effect of excessive vibration. No samples were taken for analysis or cylinders fabricated but cores were drilled as in the other series. In this series the vibrators were placed on opposite sides of the forms at the same elevation and moved up with the level of the concrete as the form was filled. By this method the form was given considerably more vibration than in the case of any other series.

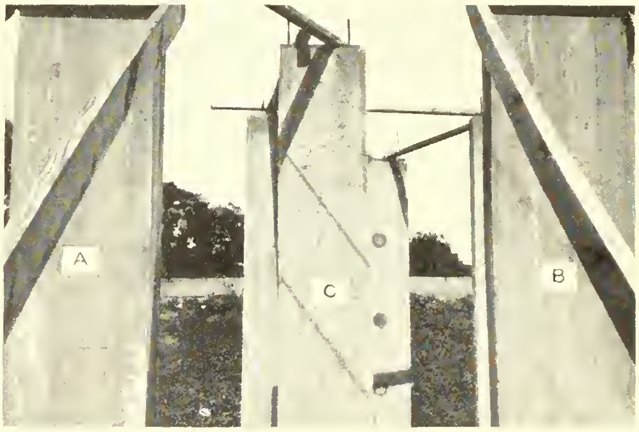


Figure 6.—Showing Pitted Surface on Top of Center Column, a Condition Typical of Vibrated Columns. Columns to Left and Right Spaded

The concrete for analysis was taken to the laboratory and the water-cement ratio and cement aggregate ratio determined by the method described by Prof. W. M. Dunagan in the Journal of the American Concrete Institute, December, 1929. The cylinders were made and stored adjacent to the columns. After 24 hours both the column and cylinder forms were removed, and no further curing provided.

At an approximate age of 4 months the columns were placed in a horizontal position and cores drilled at the points indicated in Figure 2.

All cylinders and cores were then capped and stored in the open until an age of 6 months, when they were tested for compression after immersing in water for 24 hours. Both cylinders and cores were tested in a 200,000-pound universal testing machine. The cores were approximately 5.7 inches in diameter and 8 inches high and a correction factor was applied to make the strength correspond to a specimen having a height equal to twice its diameter.

RESULTS OF INVESTIGATION DISCUSSED

Segregation determined by analysis of fresh concrete.—In Table 5 will be found the volumes of cement to total combined aggregates and the water-cement ratios for each sample of concrete taken from the forms before setting. The method used in making these determinations, as proposed by Professor Dunagan, consists of weighing the sample of concrete in air and in water, then separating the cement, fine and coarse aggregate, by washing through suitable screens and reweighing each in water. From these weights and the specific gravities of the materials, the weights in air are computed. The sum of these weights subtracted from the original weight gives the weight of water. Knowing the weight of water, cement, fine and coarse aggregate, the mix may be readily computed. In these tests no attempt was made to separate the fine and coarse aggregate so that the mix determinations are given as one part of cement to parts of combined fine and coarse aggregate by volume. A No. 100 sieve was used to separate the cement from the aggregate, correction being made for the percentage of cement retained and percentage of sand passing the same sieve. In working with the fine and coarse aggregate combined, it was necessary to use one specific gravity. Inasmuch as the gravity of the sand was somewhat higher than that of the gravel, a weighted value was used assuming the sand and gravel to be present in the proportion of 1 to 2.

The values given in Table 5 are shown for the 6 and 9 foot columns in Figure 7 and for the 12-foot columns in Figure 8. Values for the 3-foot columns are not shown in plotted form.

Table 5.—*Segregation as determined by analysis of fresh concrete*

Sample No.	Parts of total combined aggregate to 1 part of cement								Water-cement ratio, cubic feet of water to 1 sack of cement							
	Series								Series							
	A	B	C	D	E	F	G	H	A	B	C	D	E	F	G	H
3-FOOT COLUMNS																
1	5.2	5.4	5.4	4.8	4.8	5.3	5.6	6.2	0.85	1.00	0.83	0.85	0.81	0.89	0.93	0.80
2	7.6	5.8	5.9	5.6	5.9	6.2	6.2	6.2	.98	.96	.88	.83	.87	.95	.91	.84
3	5.4	5.8	6.2	6.4	6.2	6.4	5.1	5.8	.76	.91	.87	.85	.91	.96	.82	.79
6-FOOT COLUMNS																
4	5.2	4.8	5.2	4.6	5.3	5.3	----	6.2	.82	.98	.83	.79	.87	.91	----	.86
5	6.0	5.8	6.0	5.9	6.0	5.7	----	6.1	.82	.93	.97	.81	.81	.87	----	.89
6	5.7	5.8	5.4	5.7	5.5	6.1	6.2	5.0	.79	1.02	.77	.84	.83	.95	.91	.68
7	5.7	5.8	5.5	5.8	6.0	6.0	5.9	5.6	.79	.92	.83	.83	.83	.83	.87	.79
9-FOOT COLUMNS																
8	5.3	4.7	5.5	5.0	5.5	5.3	5.7	6.0	.80	.92	.83	.84	.86	.93	.83	.79
9	5.3	5.6	5.1	5.5	5.9	6.1	5.8	6.5	.79	.93	.84	.86	.84	.92	.88	.84
10	5.5	5.6	5.3	5.4	5.3	5.4	6.1	5.8	.77	.96	.85	.82	.79	.84	.86	.70
11	5.6	5.1	5.4	5.7	5.3	5.7	6.1	5.1	.82	.83	.82	.82	.75	.92	.79	.74
12	5.4	5.8	5.8	5.9	5.5	5.6	6.6	5.4	.75	.88	.86	.87	.80	.87	.86	.73
12-FOOT COLUMNS																
13	4.8	4.1	5.0	5.4	4.7	5.0	5.6	6.4	.79	.92	.79	.79	.78	.86	.85	.89
14	5.3	5.1	5.2	5.8	5.6	5.7	6.5	5.1	.81	.98	.80	.76	.83	.87	.81	.69
15	5.8	6.1	5.1	6.0	5.4	5.5	6.1	5.8	1.06	.94	.78	.95	.77	.84	.87	.71
16	5.6	5.4	5.5	5.5	5.3	5.6	6.7	6.1	.89	.85	.80	.75	.79	.86	.85	.77
17	5.8	5.9	5.3	5.6	5.4	6.1	6.3	6.3	.82	.87	.80	.76	.75	.96	.83	.80
18	5.5	5.8	5.3	5.6	5.4	5.7	5.7	6.3	.82	.91	.78	.74	.75	.89	.78	.72
19	5.4	5.6	5.4	5.7	5.7	6.0	6.0	5.9	.86	.84	.81	.76	.81	.90	.76	.68

Before discussing these data, attention should be called to the fact that individual rather than average results are shown. Each plotted point in Figures 7 and 8, therefore, represents the result of a single test on one sample of concrete taken from the indicated portion of the structure. Too great significance should not be attached to individual test results as such. Instead, an effort should be made to ascertain what, if any, general trends appear to exist, as regards (1) the relative amount of cement in terms of the volume of concrete and (2) the relative amounts of water in terms of the volume of cement (the water-cement ratio).

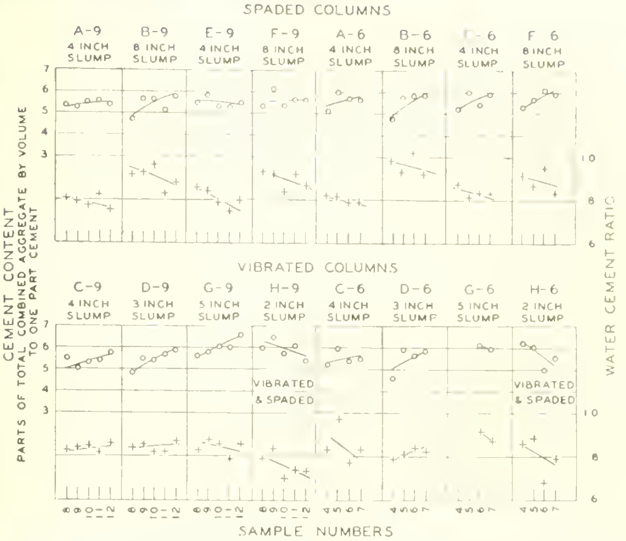


Figure 7.—Results of Tests to Determine Uniformity of Composition of Concrete in 9 and 6 Foot Columns

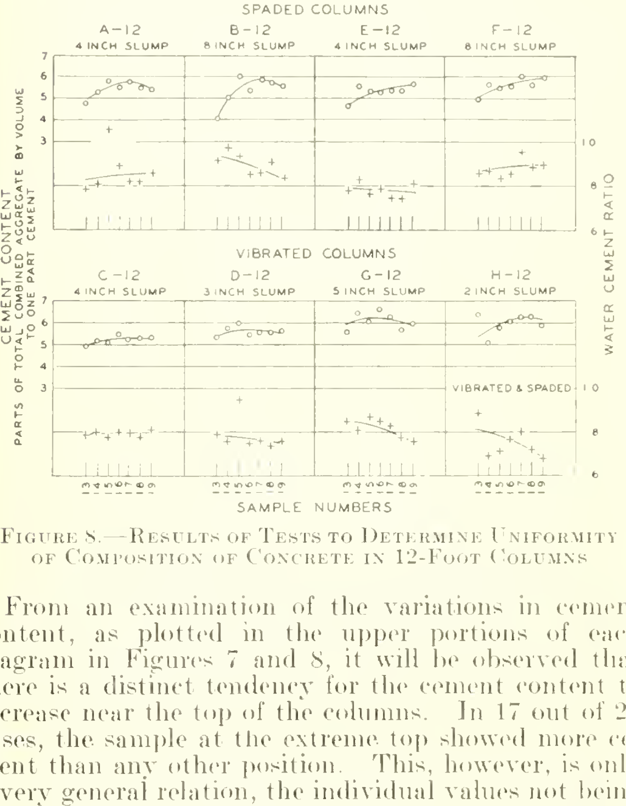


Figure 8.—Results of Tests to Determine Uniformity of Composition of Concrete in 12-Foot Columns

From an examination of the variations in cement content, as plotted in the upper portions of each diagram in Figures 7 and 8, it will be observed that there is a distinct tendency for the cement content to increase near the top of the columns. In 17 out of 24 cases, the sample at the extreme top showed more cement than any other position. This, however, is only a very general relation, the individual values not being very consistent except in a few cases, such as columns D-9 and G-9. In columns H-6 and H-9 the tendency is in the other direction. In still other cases, the individual values vary quite widely without any apparent reason. In spite of the many individual discrepancies, however, there seems to be a general tendency along the line indicated; that is, a somewhat richer mix at the top of the columns. The effect of this variation in cement content on strength is discussed in the next section of this report.

Referring now to the variations in water-cement ratios, as plotted in the lower part of each diagram in Figures 7 and 8, we find that a slight tendency toward increased water content at the top of the columns is shown in some cases. The individual results, however, vary quite widely. Here again we are confronted with the fact that each point represents but one test result, with a consequently increased tendency for experimental errors to affect the results.

It is reasonable to suppose that water gain, if occurring at all, would develop in series B and F, the two sections in which 8-inch slump concrete, spaded in the forms, was used. In Figures 7 and 8 it will be seen that for series B and F there is, with the exception of column F-12, a general tendency in all cases for water to increase toward the top. In only one case, however, F-9, is the maximum water found in the very top sample. In general, it may be said that, in so far as the segregation test was able to measure, there was no serious water gain in any of these columns. This conclusion is verified by visual inspection made during the placing of the columns. No marked water gain was noted in any case.

Segregation determined by strength tests.—As previously noted, concrete strength specimens were obtained from various portions of each column, both by molding 6 by 12 inch cylinders from samples of concrete taken from the ports and by drilling cores from corresponding locations in the columns. Tests on these specimens may also be compared to tests on samples of concrete taken just after mixing and before placing in the forms.

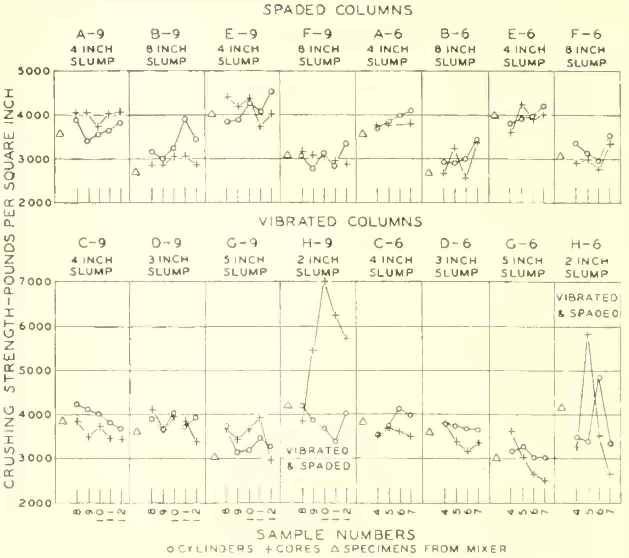


Figure 9.—Results of Tests to Determine Uniformity of Strength of Concrete in 9 and 6 Foot Columns

The results of these strength tests are shown in detail in Table 6 and are plotted for the 6, 9, and 12 foot columns in Figures 9 and 10. Here also, attention

Table 6.—*Strength tests of concrete. Values given are compressive strengths in pounds per square inch*

Specimen No.	Series A		Series B		Series C		Series D		Series E		Series F		Series G		Series H		Series K
	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cores
3-FOOT COLUMNS																	
1	3,450	3,780	2,480	3,290	4,130	4,070	3,870	3,590	3,790	3,260	2,810	2,820	3,210	3,460	4,330	4,350	------
2	3,180	3,700	2,620	2,710	3,630	3,150	3,750	3,540	3,720	3,900	2,710	3,120	3,180	3,080	3,440	3,500	------
3	3,080	3,100	2,680	2,200	3,500	3,030	3,700	3,610	3,610	3,370	2,960	2,860	3,140	2,880	4,240	3,930	------
Average	3,236	3,526	2,593	2,733	3,753	3,416	3,773	3,580	3,706	3,510	2,826	2,933	3,176	3,140	4,003	3,926	------
6-FOOT COLUMNS																	
4	3,710	3,790	2,940	2,650	3,570	3,500	3,780	3,780	3,800	3,600	3,380	2,910	3,180	3,630	3,500	3,280	------
5	3,830	3,780	2,910	3,210	3,780	3,680	3,730	3,380	3,930	4,220	3,120	3,000	3,260	3,060	3,400	5,850	------
6	4,000	------	3,000	2,550	4,150	3,620	3,660	3,160	3,990	3,920	2,980	2,780	3,000	2,640	4,890	3,510	------
7	4,100	3,800	3,410	3,380	4,000	3,500	3,650	3,350	4,210	4,000	3,520	3,360	3,040	2,500	3,370	2,680	------
Average	3,910	3,790	3,065	2,947	3,875	3,575	3,705	3,417	3,982	3,934	3,250	3,012	3,120	2,957	3,790	3,830	------
9-FOOT COLUMNS																	
8	3,930	4,010	3,180	2,870	4,230	3,840	3,900	4,100	3,850	4,400	3,080	3,160	3,730	3,650	4,200	3,860	------
9	3,410	4,040	2,980	2,870	4,110	3,500	3,670	3,630	3,900	4,170	2,790	3,080	3,140	3,420	3,870	5,440	------
10	3,560	3,760	3,250	3,020	4,030	3,740	4,010	3,960	4,250	4,370	3,110	3,010	3,190	3,670	3,690	7,040	------
11	3,670	4,010	3,920	3,040	3,800	3,460	3,720	3,830	4,080	3,710	2,840	2,910	3,460	3,910	3,360	6,220	------
12	3,840	4,070	3,420	2,850	3,680	3,430	3,930	3,370	4,550	4,000	3,350	2,870	3,270	2,950	4,040	5,710	------
Average	3,682	3,978	3,350	2,930	3,970	3,594	3,846	3,778	4,126	4,130	3,034	3,006	3,358	3,520	3,832	5,654	------
12-FOOT COLUMNS																	
13	3,960	4,070	2,740	3,080	3,950	4,120	3,850	4,280	4,310	4,100	3,050	3,250	2,990	3,170	3,710	4,770	3,920
14	3,610	4,340	2,620	2,840	3,460	2,990	4,380	4,290	4,270	4,670	2,860	3,100	3,270	3,680	4,200	5,640	3,240
15	3,820	3,950	2,870	2,650	4,000	3,570	4,170	3,860	4,680	4,500	3,360	3,210	3,110	3,070	4,060	4,320	2,900
16	3,450	3,740	2,950	2,850	4,110	3,870	4,560	3,930	4,350	4,010	3,280	3,310	2,920	3,040	3,860	4,050	2,970
17	3,320	3,580	3,010	2,760	3,910	3,930	3,430	3,720	4,450	4,140	3,420	2,980	3,490	2,790	3,620	5,380	3,460
18	3,720	3,590	2,760	2,950	3,570	3,820	3,830	3,390	4,400	4,010	3,200	3,170	3,670	2,810	3,790	5,980	4,330
19	3,650	3,780	3,250	2,820	3,800	3,800	3,960	3,330	3,820	3,690	3,780	3,040	3,500	3,050	4,150	6,670	6,630
Average	3,647	3,864	2,886	2,850	3,828	3,728	4,025	3,828	4,326	4,160	3,278	3,151	3,278	3,087	3,913	5,258	3,921
Grand average all heights	3,647	3,827	3,000	2,873	3,864	3,612	3,871	3,689	4,103	4,002	3,137	3,049	3,250	3,182	3,880	4,851	------
SPECIMENS TAKEN FROM MIXER																	
	3,450	------	2,670	------	3,800	------	3,280	------	3,630	------	2,430	------	2,930	------	4,000	------	------
	3,660	------	2,670	------	3,830	------	3,820	------	4,330	------	3,000	------	2,910	------	4,360	------	------
	------	------	------	------	3,800	------	3,610	------	3,920	------	2,670	------	3,170	------	4,180	------	------
Average	3,555	------	2,670	------	3,810	------	3,570	------	3,960	------	2,700	------	3,003	------	4,180	------	------

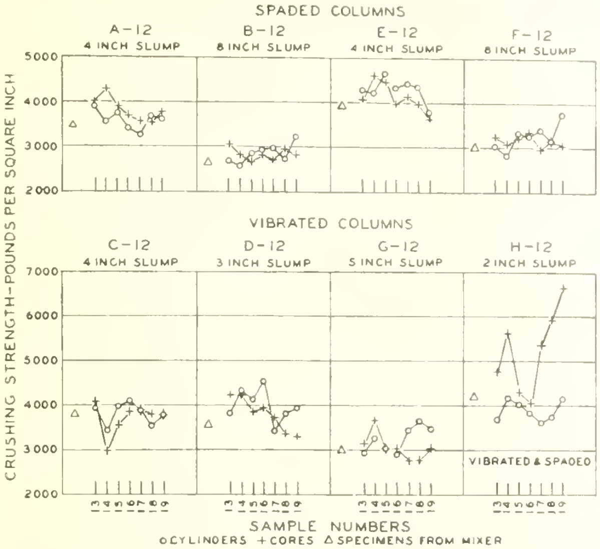


Figure 10.—Results of Tests to Determine Uniformity of Strength of Concrete in 12-Foot Columns

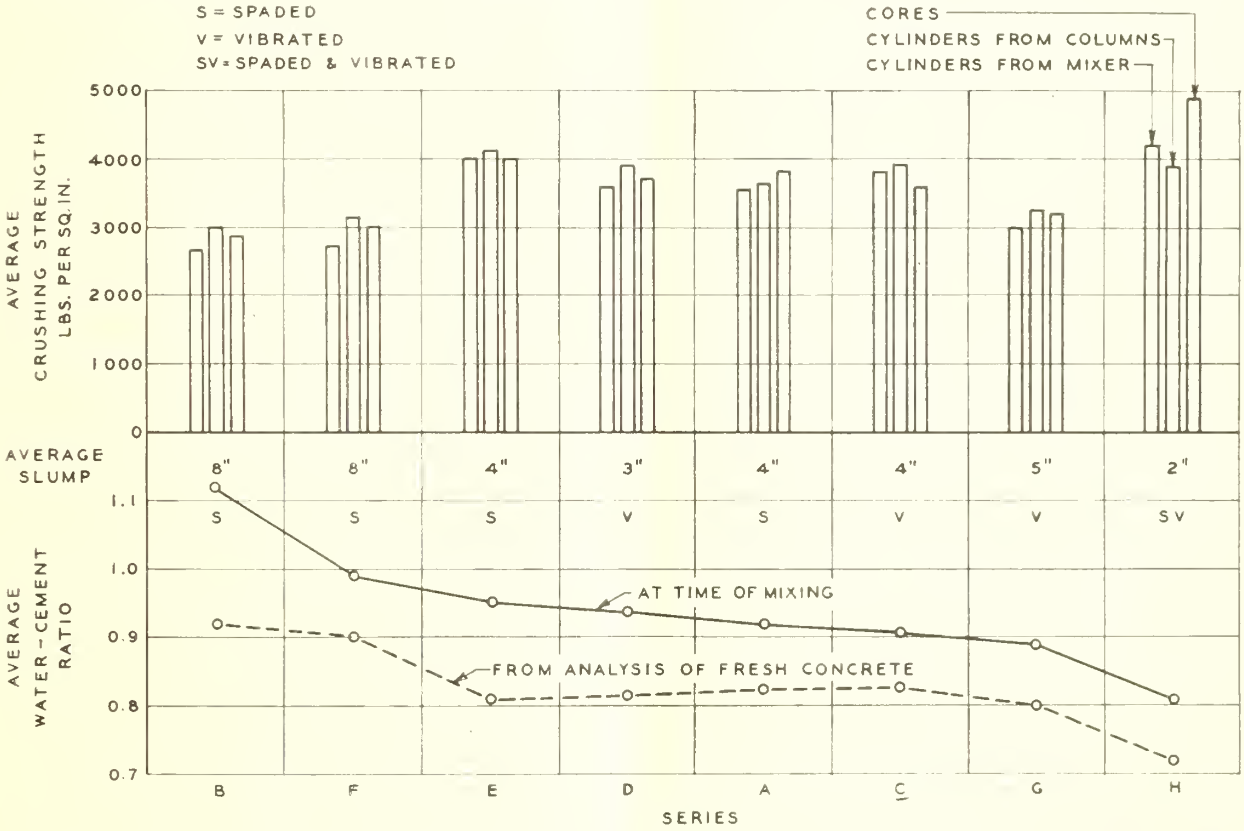


Figure 11.—Relation Between Strength of Concrete Taken From Mixer and That Taken From Columns. Figure Also Shows Water-Cement Ratios Obtained at Time of Mixing and Those Obtained From Analysis of Fresh Concrete

should be called to the fact that results of tests on individual specimens are shown and that therefore the same degree of concordance should not be expected as when average results representing a number of individual tests are given.

In spite of this, the results seem to be reasonably concordant. Attention may be called especially to the agreement between the results of tests on the cylinders as compared to the corresponding cores. In only one case, that is, series H, is there serious disagreement. In this series, in which the concrete was both vibrated and spaded, the tests on the cores gave in general much higher strengths than those on the cylinders. This series contains the driest concrete used in the experiment, 2-inch slump, and it is possible that under these conditions vibration consolidated the concrete sufficiently to increase the strength of core specimens considerably beyond the results obtained on the cylinders. In no other series did the cores from the vibrated concrete show appreciably higher results than the cylinders. As a matter of fact the average core strengths, as shown by Table 6 and in Figure 11, were somewhat lower. The conclusion is reached, therefore, that, in general, vibration did not increase the crushing strength of the concrete. Also, there appears to be no general tendency for either increased or decreased strength in the tops of the columns. This checks the results of the segregation tests and further indicates that no serious segregation developed under these conditions.

In Figure 11 are shown the average results of tests for strength as well as the average water-cement ratios for each series of columns. With the exception of the values for water-cement ratio as determined at the mixer (that is, the amount of water that went into the concrete with proper allowance for moisture in the aggregates) the data are taken from Tables 4 and 5. Each value represents the average of all of the individual determinations made on each of the four columns making up the series. With one or two exceptions, then,

(Continued on p. 72)

GASOLINE TAXES, 1931

Tax earned on motor vehicle fuel, etc., refunds, disposition of fund, and gallons taxed, during full calendar year, 1931

[From reports of State authorities]

State	Gross tax assessed prior to deduction of refund	Exemption refund (deducted from gross tax)	Net tax earning on motor vehicle fuel [1]	Other receipts, under tax law (licenses, etc.)	Grand total earning (tax and other receipts)	Disposition of grand total earning, according to law						Tax rate, 1931			Gasoline, or other fuel for motor vehicles, taxed	
						Collection and administration cost [2]	Construction and maintenance on rural roads		State and county road bond payments [4]	On city streets	For other than highway purposes	Cents per gallon		Date of rate change	Net gallons taxed	Percentage change
							State highways [3]	Local roads [3]				Jan. 1	Dec. 31			
Alabama	$7,197,474		$7,197,474		$7,197,474	$16,644	$2,204,117	$3,248,655	$1,728,058			4	5	7/28	162,670,816	−5.7
Arizona	3,595,902	$391,614	3,204,288	$32	3,204,320	(6)	2,227,256	977,064				4	5	1/30	64,701,865	−3.1
Arkansas	6,956,952	508,903	6,448,049		6,448,049	[7] 20,000	[7] 200,000	787,846	[8] 5,282,399	$107,804	[9] $50,000	5	6	2/26	110,579,175	−14.0
California	44,588,156	4,724,519	39,863,637		39,863,637	[10] 124,632	26,467,700	13,233,850			[11] 37,455	3	3		1,328,787,915	14.3
Colorado	7,059,749	805,411	6,254,338		6,254,338	[12] 67,379	4,330,871	1,670,479		185,609		4	4		156,358,455	1.8
Connecticut	4,732,734	[13] 4,741	4,727,993	48,759	4,776,752	(14)	4,776,752					2	2		236,399,661	5.9
Delaware	1,141,921	69,860	1,072,061		1,072,061	(15)	533,881		[16] 538,180			3	3		35,735,365	5.8
Florida	14,986,170		14,986,170	32,050	15,018,220	[17] 32,050	5,781,048	423,555	[18] 5,781,047		[19] 3,000,520	6	7	8/1	235,057,035	3.5
Georgia	13,313,500		13,313,500		13,313,500	4,200	8,872,867	2,218,217			[20] 2,218,216	6	6		221,891,668	−0.6
Idaho	2,953,992	355,626	2,598,366	[21] 11,061	2,609,427	[22] 30,355	2,325,833		[23] 242,594		[24] 10,645	5	5		[25] 51,967,321	−4.5
Illinois	30,495,596	1,429,911	29,065,685		29,065,685	106,834	19,305,901	9,652,950	(26)			3	3		998,856,165	5.8
Indiana	19,258,819	1,224,266	18,034,553	8,972	18,043,525	66,942	13,482,437	3,370,609		1,123,537		4	4		450,863,830	5.1
Iowa	12,378,724	1,451,135	10,927,589		10,927,589	62,956	6,372,372	4,492,261				3	3		364,252,984	3.2
Kansas	11,383,674	3,312,789	8,070,885		8,070,885	(27)	6,270,885	1,800,000				3	3		269,029,495	−11.5
Kentucky	8,810,130		8,810,130	2,864	8,812,994	30,987	8,782,007					5	5		176,202,606	4.7
Louisiana	9,398,107	324	9,397,783		9,397,783	62,000	4,791,951		2,664,276		[28] 1,879,556	5	5		187,955,663	1.7
Maine	4,609,316	226,588	4,382,728	[29] 56,647	4,439,375	32,772	2,203,302	2,203,301				4	4		109,568,209	6.6
Maryland	7,796,063	365,061	7,431,002		7,431,002	20,549	5,868,362			1,467,091	[30] 75,000	4	4		185,775,062	6.3
Massachusetts	15,573,815	267,439	15,306,376		15,306,376	(31)	12,535,626	902,267		1,868,483		2	3	5/1	558,555,950	5.8
Michigan	23,908,116	2,075,769	21,832,347	[32] 40,293	21,872,640	54,639	14,707,047	4,070,661	3,000,000	(33)	[34] 40,293	3	3		727,744,907	0.7
Minnesota	12,542,814	1,472,655	11,070,159		11,070,159	(35)	7,380,106	3,690,053				3	3		369,005,304	6.9
Mississippi	5,882,264		5,882,264	[36] 127,172	6,009,436	[37] 11,281	2,901,625	2,871,310			[38] 225,220	5	5½	11/1	115,640,338	−14.9
Missouri	9,475,871	269,307	9,206,564		9,206,564	49,189	9,157,375		(39)			2	2		460,328,204	6.6
Montana	3,801,888	783,734	3,018,154		3,018,154	27,147	2,970,346		20,661			5	5		60,363,076	2.6
Nebraska	9,176,004	79,756	9,096,248		9,096,248	11,250	6,813,749	2,271,249				4	4		227,406,207	0.4
Nevada	905,533	127,615	777,918		777,918	(40)	777,918					4	4		19,447,944	15.2
New Hampshire	2,720,395	63,252	2,657,143		2,657,143	(41)	1,992,857		664,286			4	4		66,428,585	6.3
New Jersey	17,141,415	16,783	17,124,632	48,215	17,172,847	18,600	11,636,747		427,500	5,000,000	[42] 90,000	3	3		570,821,076	4.4
New Mexico	2,664,704		2,664,704	20,873	2,685,577	[43] 60,110	1,686,034		928,933		[44] 10,500	5	5		53,294,084	−2.0
New York	31,422,861	878,800	30,544,061	45,095	30,589,156	[45] 50,000	22,904,367	6,107,831			[46] 1,526,958	2	2		1,527,203,055	6.2
North Carolina	14,371,270	346,967	14,024,303		14,024,303	9,370	4,379,478	2,639,474	[47] 6,400,509		[48] 595,472	5	6	4/1	249,609,024	0.4
North Dakota	3,149,024	1,118,786	2,030,238	2,217	2,032,455	[49] 33,237	1,332,351	666,175			[50] 692	3	3		67,674,591	3.1
Ohio	41,272,028	1,943,975	39,328,053		39,328,053	107,956	22,061,305	9,805,024		7,353,768		4	4		983,201,323	6.1
Oklahoma	11,674,218	8,786	11,665,432		11,665,432	71,552	7,249,008	2,416,302			[51] 1,928,570	4	5	3/25	252,483,145	−16.5
Oregon	6,935,821	748,903	6,186,918		6,186,918	18,300	6,168,618		(52)			4	4		[53] 155,032,787	0.05
Pennsylvania	32,929,982	477,305	32,452,677	[54] 735,913	33,188,590	[55] 765,125	23,423,209	5,563,743	3,436,513			3	3		1,081,755,912	16.5
Rhode Island	1,963,128	70,493	1,892,635	3,640	1,896,275	(56)	1,422,206		474,069			2	2		94,631,763	9.3
South Carolina	7,274,440	28,451	7,245,989		7,245,989	(57)	4,796,512	1,207,665	[58] 1,241,812			6	6		120,766,492	1.4
South Dakota	5,157,175	1,762,500	3,394,675		3,394,675	[59] 48,602	3,346,073		(60)			4	4		84,866,877	−3.1
Tennessee	11,461,023		11,461,023		11,461,023	57,305	4,209,658	2,994,786	[61] 4,199,274			5	[62] 7	12/19	206,707,098	−3.6
Texas	33,462,890	2,948,332	30,514,558		30,514,558		22,885,918				[63] 7,628,640	4	4		762,863,942	3.3
Utah	2,311,734	2,507	2,309,227	451	2,309,678	4,214	2,305,464		(52)			3½	4	5/12	60,362,698	0.4
Vermont	1,966,544		1,966,544		1,966,544	(64)	1,562,638		278,906		[65] 125,000	4	4		49,163,593	4.6
Virginia	12,210,867	765,652	11,445,215		11,445,215	(66)	8,011,651	3,433,564				5	5		228,904,309	6.2
Washington	12,313,127	1,280,665	11,032,462		11,032,462	(67)	8,587,166	2,445,296				3	5	4/1	244,529,589	1.1
West Virginia	5,665,218	278,001	5,387,217	8,005	5,395,222	14,777	2,054,573		3,325,872			4	4		134,680,421	0.5
Wisconsin	16,471,048	690,867	15,780,181	(68)	15,780,181	20,469	8,830,273	4,514,493	[69] 1,797,646	617,300		2	4	4/1	431,504,707	3.8
Wyoming	1,587,014		1,587,014		1,587,014	5,894	1,129,841	395,279	56,000			4	4		39,675,361	9.7
District of Columbia	1,740,022	13,726	1,726,296		1,726,296					1,726,296		2	2		86,314,800	7.9
Total			536,397,458	1,192,259	537,589,717	2,117,317	354,017,281	100,073,959	[70] 42,488,535	19,449,888	19,442,737	Weighted average rate, 3.48 cents			15,407,650,452	4.4

[1] Net gasoline tax earned after deduction of refunds allowed by law.

[2] Many States pay collection cost from other State funds, and such are noted. Administration cost here includes balances allocated to reserve funds for administrative purposes, and amounts are noted.

[3] Since this table covers the calendar year earnings, but not the actual collections during the year, these columns are not comparable with similar columns in the financial tables, F-1 and F-4, issued by this bureau, which cover different periods fixed by State and local agencies; also certain funds are allocated to bond payments shown in next column.

[4] Payments are for State highway bonds, except as noted.

[5] Shows per cent increase or decrease (−) compared to net gallons reported in previous year.

[6] Paid from State highway budget, $16,556.

[7] Estimates reported.

[8] Includes payments on county bonds, $3,395,828.

[9] For expenses of motor-vehicle department.

[10] Includes $61,857 for next 6 months' expenses, in reserve.

[11] For expenses of transportation license tax division.

[12] Includes all expenses of State inspector of oils, and special expenses for shale-oil investigation in cooperation with U. S. Bureau of Mines.

[13] Excludes tax on 11,254,626 gallons exempted when purchased.

[14] Expenses of $30,009 paid from motor-vehicle fees.

[15] Paid from State treasury funds.

[16] Includes $349,355 as payments on county road bonds.

[17] Includes $11,690 gasoline tax reserve fund, assigned to administration.

[18] Payments on county road bonds.

[19] Consists of $200,000 to permanent fund for State buildings of higher learning, $1,917,773 for county schools, and $882,747 to State general revenue fund.

[20] To an equalization fund for public schools.

[21] Includes $10,645 collected on 212,905 gallons sold for airplanes.

[22] Includes $17,007 to reserve for refunds.

[23] To State treasury note redemption fund.

[24] To an aviation fund, being amount of aviation gas tax.

[25] Excludes 212,905 gallons sold to airplanes and taxed.

[26] County bond payments from gasoline tax included in local roads.

[27] Paid from State general fund, $15,000.

[28] Consists of $939,778 for State board of eduction, and $939,778 for boards of commissioners of ports of New Orleans and Lake Charles Harbor.

[29] Consists of 1 cent tax on all sales of gasoline not used by motor vehicles.

[30] To conservation department for oyster propagation.

[31] Paid from State general fund, $22,500.

[32] Consists of taxes on gas used in aeronautics and aeronautic licenses.

[33] Amount for city streets reported under State highways.

[34] Consists of $38,108 from tax on gas used in aeronautics assigned to State aeronautic fund, and $2,185 collected from licenses to State general fund.

[35] Paid from State general fund, $13,000.

[36] Special taxes collected in two counties for sea-wall financing.

[37] Includes $5,881 to State accounting department for auditing.

[38] For sea-wall protecting road. Derived from extra gas taxes in Harrison County (2 cents) and Hancock County (3 cents) shown in other receipts, $127,172; and the remainder from State highway system share of gas tax receipts.

[39] Amount assignable to sinking fund for bond payments, included in State highway fund column.

[40] Paid by tax commission which collects taxes: Amount not reported.

[41] Paid by motor-vehicle department: Amount not reported.

[42] For inland waterways under Department of Commerce and Navigation.

[43] Includes balances in suspense fund and operating fund of $28,318.

[44] Loans to motor-vehicle department and public-auditing department.

[45] Paid from State general fund, $75,463. Amount shown is for reserve to pay refunds.

[46] To New York City general fund.

[47] Includes $1,381,146 payments on county road bonds.

[48] For State highway patrol and administration expenses of State revenue department.

[49] Includes $8,237 to reserve for refunds.

[50] Dealers' license fees credited to State general fund.

[51] Consists of $1,000,000 special emergency relief for destitute, and remainder for distribution of field and garden seed.

[52] Payments reported from motor-vehicle fees, instead of assigning a pro rata share to gasoline taxes.

[53] Includes approximately 3,118,610 gallons of distillate taxed at 3½ cents per gallon.

[54] Including expenses of department of revenues prorated to gasoline taxes.

[55] Consists of delinquent collections of previous years, penalties, and fines.

[56] Paid from motor-vehicle fees, $15,737.

[57] Paid from State tax commission appropriation. Amount not reported.

[58] Includes all payments on State highway bonds, as pro rata share not reported in disposition of motor-vehicle receipts.

[59] Includes $31,602 to reserve for refunds.

[60] All State highway bonds retired on Jan. 15, 1931, by funds previously accumulated.

[61] Includes county road bond payments $2,089,184.

[62] A tax of 6 cents was effective July 1, 1931 to Dec. 19, 1931.

[63] For free-school fund.

[64] Paid from motor-vehicle department appropriation, estimated $2,000.

[65] Transfer to motor-vehicle department.

[66] Paid by State appropriation, $14,096.

[67] Paid from motor-vehicle fund, $10,000.

[68] Dealers' license fees paid into general fund.

[69] Payments on county road bonds.

[70] Includes payments on county road bonds, $27,451,735, on State highway bonds, $14,794,206, and in note fund, $242,594.

(Continued from p. 63)

50 feet to the right of the center line at station 17+00 before drainage revealed a depth of muck of 6 feet. Nine months later this had consolidated to 4.75 feet. This represents a loss of 54,450 cubic feet per acre.

Figure 8 shows a series of photographs taken at or near station 10+00 at different stages of the work, from the construction of a surface drainage ditch to the completion of the road surface.

As a result of the drainage this soil, which a few months before failed to support the power shovel on rafts, carried heavily loaded material trucks and grading was completed without difficulty.

Thus, this soil was improved by the very means which might have been the least effective under other circumstances.

The method employed was successful in this instance only because of the peculiar topographic features encountered in combination with certain physical properties possessed to such a marked degree by the muck soil. The decision as to proper methods of stabilization was possible only as a result of thorough knowledge of all the factors involved.

(Continued from p. 69)

each value represents the average of nineteen individual tests. This chart indicates quite clearly that the crushing strength of the concrete as determined by cylinder tests on samples taken as the mixer was discharged approximated very closely the actual average strength of the concrete in the column, as determined either by molding cylinders of concrete taken from the forms or by means of cores drilled from the hardened concrete.

The data also indicate that the average net water-cement ratio as determined by the Dunagan analysis approximately one hour after the concrete had been placed was about 0.1 less than the net water-cement ratio at the time of mixing.

The results of strength tests of cores on the one 12-foot column (series K) are shown in Figure 12. As previously explained this column was vibrated, but in a more severe manner than any other series. The concrete going into the bottom of this form (represented by core 19) appeared dry, so the consistency was changed from a 2-inch slump to a 4-inch slump after about 2 feet of concrete had been placed. While this change in water-cement ratio would naturally result in a higher strength at the bottom of the column than at the top, the difference in strength is greater than would be expected from this cause alone. The severe vibration is probably responsible for some of this difference for it will be observed that the strength increases from a point 9 feet from the bottom to a maximum at a point 1 foot from the bottom. It also increases from the 9-foot mark as the top is approached. In series H the curve for the 12-foot column has the same general trend. (See fig. 10.) It would appear from the above that the strength of the concrete is not appreciably affected by compacting it either by spading or moderate vibration as was done on series A to series G, inclusive. When the two methods are combined, or when the vibration is of a more severe nature, as in series H and series K, a marked effect is produced.

CONCLUSIONS

The conclusions derived from this study are naturally restricted in their application to conditions comparable to those under which the tests were conducted. These conditions include a range in slump from 2 to 8 inches; placing by both spading and vibratory methods; forms 8 inches in width; and a fine aggregate of relatively fine sand. The conclusions are:

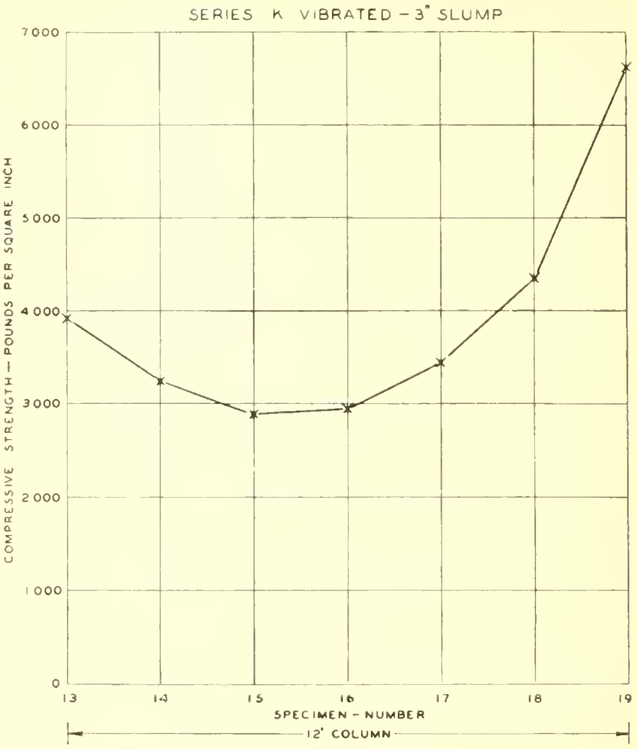


Figure 12.—Compressive Strength of Cores Obtained From Series K, Which was Subjected to Severe Vibration

1. No serious segregation of concrete or water gain took place. This conclusion applies for a range in slump varying from 2 to 8 inches and for both the spading method and the vibratory method of placing. In this connection, attention is called to the fact that forms only 8 inches in width were employed in these tests and also that a relatively fine sand was used. Both of these factors probably contributed considerably to the absence of serious segregation.

2. With the exception of the one series of tests in which the concrete was both spaded and vibrated (series H), the method of placing by vibration produced concrete having about the same average strength and uniformity as similar concrete placed by spading.

3. A combination of spading and vibration appeared to increase the strength of the concrete considerably beyond the strength obtained by either spading or vibrating alone.

4. With the exception of one series of tests (series H), the average strength of concrete as determined by cores drilled from the columns was approximately the same as the strength of the concrete determined by tests on samples taken as the mixer was discharged.

5. The net water-cement ratio taken approximately one hour after the concrete had been placed averaged about 0.1 less than the net water-cement ratio at time of mixing.

The bureau wishes to acknowledge the courtesy of the Electric Tamper & Equipment Co., Ludington, Mich., for the loan and operation of the electric vibrators used in the work.

ROAD PUBLICATIONS of the BUREAU OF PUBLIC ROADS

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.

Report of the Chief of the Bureau of Public Roads, 1925.

Report of the Chief of the Bureau of Public Roads, 1927.

Report of the Chief of the Bureau of Public Roads, 1928.

Report of the Chief of the Bureau of Public Roads, 1929.

DEPARTMENT BULLETINS

*No. 136D . . Highway Bonds. 20 cents.

*No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

*No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

*No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

*No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.

No. 1486D . . Highway Bridge Location.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys.

No. 265T . . Electrical Equipment on Movable Bridges.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications Contract Forms, and Estimates for Federal-Aid Highway Projects.

*No. 93MC . . Direct Production Costs of Broken Stone. 25c

No. 109MC . . Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATION

No. 76MP . . The Results of Physical Tests of Road-Building Rock.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927.)

Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States. (1930.)

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, was printed in PUBLIC ROADS, vol. 13, No. 3, May, 1932. Copies of this list may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MAY 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
				Initial	Stage [1]	Total			Initial	Stage [1]	Total		
Alabama	2.393.9	$ 1.812,011.04	$ 884,109.47	74.1		74.1	$	$				$ 5,469.694.24	Alabama
Arizona	1.232.8	2.367,513.11	1,701,787.22	117.1	27.9	145.0	513,875.38	386,896.72	8.9	19.4	28.3	852.053.43	Arizona
Arkansas	1.960.1	2.537.870.43	1.169,880.01	64.8	45.3	110.1	395.313.38	197.045.12	17.3		17.3	2,144.072.23	Arkansas
California	2.301.1	10.489,263.29	4.781.591.52	224.3	38.6	262.9	3.005,968.60	1,345,216.58	53.9	17.0	70.9	151,622.60	California
Colorado	1.583.4	4,457,020.37	2,331,213.18	183.5	50.3	233.8	203,412.66	114,236.51	8.1	.5	8.6	2.097.113.09	Colorado
Connecticut	282.0	4.034.661.02	1,467,917.99	29.4		29.4	563.726.40	263,977.96	7.7		7.7	450.266.59	Connecticut
Delaware	345.2	808,757.25	404,378.62	34.0		34.0	162.192.00	81.096.00	12.4		12.4	167.140.82	Delaware
Florida	617.5	4,232,614.05	1.935.392.29	111.2		111.2	410,013.79	205.006.88	14.9		14.9	2,797.004.58	Florida
Georgia	3,062.8	6,488,979.48	3.031.546.68	153.5	171.2	324.7	800.640.68	397.410.33	14.5	22.2	36.7	1,284,326.84	Georgia
Idaho	1,474.4	2,301,263.46	1.336.548.68	117.4	80.7	198.1	912.173.80	551.669.33	19.9	164.2	184.1	753.666.09	Idaho
Illinois	2,640.2	25,039,271.01	11,663,206.91	748.9	32.8	781.7	4.558.209.19	1,933,042.38	146.9	8.8	155.7	1.224.027.26	Illinois
Indiana	1,788.5	10,089,248.33	5.001.315.41	324.4	16.1	340.5	2.928,955.25	1,447,252.23	139.7	4.4	144.1	277.921.19	Indiana
Iowa	3.376.4	2.550.856.28	1.190.093.28	148.6	14.2	162.8	2.428.843.53	1,141,055.93	107.1	39.8	146.9	468.655.99	Iowa
Kansas	3.670.8	4.815,729.58	2.353.680.03	292.8	29.9	322.7	2,300,589.40	1.112.906.07	85.9	228.6	314.5	871.135.30	Kansas
Kentucky	1.885.0	2.553.780.56	1,132.511.46	137.9	4.0	141.9	1.241,059.97	519.790.62	43.1	84.9	128.0	1.337.204.72	Kentucky
Louisiana	1.566.7	7.471.657.46	3.511.759.18	86.9	10.6	97.5						847.317.82	Louisiana
Maine	708.4	2.504.257.11	1.062.976.98	50.4		50.4	1,940,190.50	895.232.96	75.7		75.7	162.080.43	Maine
Maryland	776.8	590,183.00	223,974.80	20.5		20.5	1.222.453.98	573.117.51	41.7	6.2	47.9	187.216.51	Maryland
Massachusetts	814.8	7.767.729.70	3.206,471.58	73.7	.1	73.8						566,527.21	Massachusetts
Michigan	2,048.5	8.573.575.70	3.934,374.90	393.4	33.1	426.5	691,240.18	336.705.00	35.6	7.5	43.1	2,859.371.63	Michigan
Minnesota	4,031.9	8.104,776.42	2.713.578.48	162.0	216.0	378.0	4,022.137.30	1.265,722.90	58.6	101.0	159.6		Minnesota
Mississippi	1.797.0	4.157.313.62	2,039,029.74	170.3	75.4	245.7	44.671.02	22,335.51	6.4		6.4	5.210.985.59	Mississippi
Missouri	2.960.4	4,545,016.92	2.036.567.15	144.6	38.0	182.6	3.738.561.17	1,546,441.89	80.3	62.2	142.5	513.591.48	Missouri
Montana	2.704.1	4,660,149.39	2.620,258.74	403.1	31.5	434.6	182,379.80	105,440.57	20.5	4.2	24.7	3.443,744.79	Montana
Nebraska	4.234.6	3.380.123.81	1.678,560.14	159.9	40.4	200.3	327.120.69	163.560.33	11.5	4.5	16.0	2.538.206.45	Nebraska
Nevada	1.291.4	1,633,286.93	1.317.427.96	38.7	135.3	174.0	851.343.37	686.563.38	3.3	54.4	57.7	266,079.89	Nevada
New Hampshire	418.4	625.542.30	246.912.44	12,1	2.5	14.6						553.268.42	New Hampshire
New Jersey	605.7	3,760,915.32	1.327,092.02	36.3	.2	36.5						1,592,693.57	New Jersey
New Mexico	2.222.8	1.942.277.87	1,275,777.00	89.3	17.7	107.0	1.016.475.46	647.378.40	41.1	6.5	47.6	883,167.09	New Mexico
New York	3.265.8	13,180,149.00	5.945,419.50	306.7		306.7	6,004.600.00	2.322.550.00	117.1		117.1	2.849.008.54	New York
North Carolina	2,222.8	1,386.104.89	670,155.41	76,1		76.1	338.402.82	169.201.39	7.1	9.0	16.1	3.710,893.71	North Carolina
North Dakota	5.017.6	3.258.451.21	1.631,733.01	353.7	249.1	602.8	1,920.049.65	978.007.79	189.8	448,2	638.0	1.187,254.68	North Dakota
Ohio	2.854.7	6,477,066.69	2.284,512.94	97.2	18.4	115.6	6,376,445.01	2,324.838.24	150.4	51.8	202.2	1,894.280.38	Ohio
Oklahoma	2,266.5	4.127,708.92	1,976.092.57	164.5	50.1	214.6	1,097.682.01	553,645.37	84.3	6.8	91.1	1,629.144.80	Oklahoma
Oregon	1,519.5	4.384,818.51	2.451,443.55	131.4	47.1	178.5	1.279.991.81	698,815.63	29.1	56.2	85.3	1.031.724.38	Oregon
Pennsylvania	3.012.6	2.011,139.61	824.066.79	48.8		48.8	7,548.654.14	3.467,301.85	259.2		259.2	1.657.527.60	Pennsylvania
Rhode Island	255.4	895.613.98	443,694.82	16.6	2.2	18.8	122,356.08	61.178.04	.2		.2	301.519.98	Rhode Island
South Carolina	2,022.6	2.620.162.19	1.154.390.02	55.1	50.7	105.8	337,792.67	168,242.45	18.1		18.1	881,417.68	South Carolina
South Dakota	4.028.4	3.495.987.17	1.976.474.77	239.9	181.9	421.8	276,819.41	191.436.52	49.0	6.7	55.7	1.164,012.22	South Dakota
Tennessee	1,678.6	1,363,759.03	665.020.93	52.5	6.4	58.9	413,992.12	206.996.03	4.5	5.5	10.0	3.376.953.33	Tennessee
Texas	7,721.9	14.491.718.55	6.595.600.93	686.5	199.4	885.9	4.602,766.32	2.085.024.90	252.0	217.9	469.9	3,853.134.14	Texas
Utah	1,213.7	1,114,052.85	799,096.25	91.4	6.6	98.0	669.624.92	495,522.31	19.4	107.6	127.0	750.154.84	Utah
Vermont	339.1	1.244.646.56	568.891.61	39.2		39.2	171.773.36	65,419.72	6.5		6.5		Vermont
Virginia	1.925.9	1,880,674.52	940.337.19	108.4	.1	108.5	317.554.02	151,912.13	14.5	8.7	23.2	2,003.328.75	Virginia
Washington	1.183.1	3.031.135.35	1,495,394.35	106.4	17.7	124.1	326,289.53	176.000.00	12.0		12.0	1.170.143.83	Washington
West Virginia	889.8	1.679.821.81	720,327.72	37.8	10.5	48.3	548.482.63	261,406.32	16.4		16.4	978.250.89	West Virginia
Wisconsin	2,671.2	4,503,175.61	1,890,331.85	154.3	9.6	163.9	2,046.550.97	822.086.27	27.8	45.1	72.9	1.164.379.63	Wisconsin
Wyoming	1,937.2	3,395.825.03	2,065.905.41	225.1	155.5	380.6	448.120.95	288.692.67	16.5	60.9	77.4	244.716.57	Wyoming
Hawaii	76.3	809,346.39	347.057.35	22.0		22.0	422,233.57	417.721.92	6.7	3.6	10.3	1.564.725.89	Hawaii

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 5 ▽ JULY, 1932

ON THE MOUNT VERNON MEMORIAL HIGHWAY

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 5

July, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE DESIGN OF STREET AND HIGHWAY INTERSECTIONS

By L. S. TUTTLE and E. H. HOLMES, Assistant Highway Economists, United States Bureau of Public Roads[a]

TWO general methods of solving the problems and difficulties created by the increase in volume and speed of traffic have been developed. One method of control regulates traffic movement by such artificial means as driving rules, city ordinances, traffic officers, or signal devices, while the other seeks to minimize the necessity of artificial restrictive measures through the physical design of streets and highways for the free flow of traffic.

This paper deals with two classes of intersections, those where traffic volume necessitates regulation and those where faulty design requires it. It is proposed to show how in certain instances intersections may be developed which will obviate all control devices and rely solely upon the physical construction of the various details to induce drivers to take the proper course voluntarily, to give preference to the major traffic without too severely penalizing the minor traffic, and, when some control device is absolutely necessary, to facilitate its operation by the proper physical design.

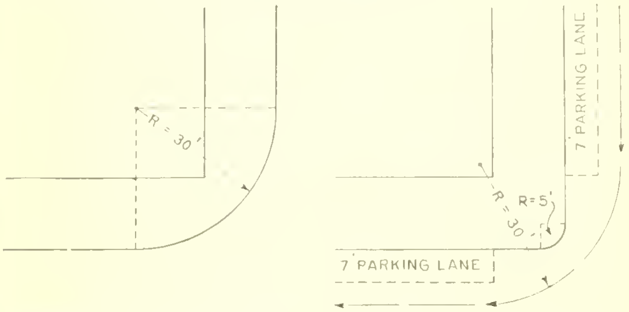
INCREASED EFFICIENCY FOR EXISTING INTERSECTIONS THROUGH MINOR CHANGES IN PHYSICAL LAYOUT

Curb radii.—A detail warranting most careful consideration in intersection design is the radius of the curves rounding off the curbs of the intersecting streets. Cutting back the curbs at existing intersections to facilitate the right turn movement is a simple and effective device for increasing intersection efficiency, particularly where the percentage of right turns is large.

A proper curb radius is one which can be followed by the vehicle having the largest normal inside turning radius. The largest radius which will be required for any but an exceptional passenger car is under 25 feet, but various types of trucks and busses have larger inside turning radii. It should not be necessary to provide for an occasional vehicle having a 40-foot turning radius, but it is absolutely necessary for efficient intersection operation that the radius be sufficient to allow the normal vehicle to follow the curb in turning and, for this reason, a curb radius of 30 feet is recommended as a minimum for highway intersections and is incorporated in all the designs presented.

For street intersections this rule applies only when the vehicle making the turn approaches in the outside lane of the roadway. Such would not be the case if parking were permitted along the intersecting streets and, while a 30-foot radius would be entirely satisfactory from the traffic viewpoint in this case, it may unnecessarily restrict sidewalk area. In this instance the proper turning radius depends not only on the vehicular requirements, but also on the sidewalk width and parking restrictions. If parking is restricted for a distance back of the intersection to allow another lane of traffic to collect while waiting for a signal, then the radius should be the same as for an open highway intersection, but in the case of the intersection of minor streets where there is no control and consequently no accumulated stoppage, parking should not be restricted so severely and so a design for curb rounding as shown in Figure 1 should be developed.

The above discussion concerns only the minimum turning radius of vehicles and results in a turn which restricts the speed to approximately 12 miles per hour. Obviously, in the case of an outlying intersection where land is available, it would be desirable to increase this turning radius to some figure which would permit higher speeds. This step becomes important at heavily traveled intersections. If the speed which would be maintained provided there were no turning traffic is 20 miles per hour and the radius of the curb of the intersection is such that only 12 miles per hour can be maintained in the turning, turning traffic will unnecessarily retard through traffic, materially reducing the capacity of the intersection. Therefore, it is advisable to design the turning radius to accommodate the same speed which would be maintained by traffic moving straight through the intersection, except where a heavy pedestrian movement will control the turning speed.



ROUNDED FOR PROPER TURNING RADIUS, SERIOUSLY DECREASES SIDEWALK WIDTH

SHORT RADIUS CURB; EFFECTIVE WHERE PARKING IS PERMITTED.

FIGURE 1.—DESIGN OF SHORT-RADIUS CURB

The speed which will be maintained around a curve will vary with the individual driver, but usually will not exceed that at which passengers begin to feel discomfort. This discomfort results from the effort to maintain an erect position in overcoming the centrifugal force resulting from the deflection from a straight course.

In order to determine the uncomfortable rate of deflection a series of test runs were made around curves of various radii in a coupé and also in a sedan, both in the lower price range. When it became necessary for the occupants to exert a distinct effort to remain erect, a speedometer reading was taken. While this method was rough, the observed speeds plotted against the radii gave a remarkably smooth curve, the two cars showing close agreement. Theoretically the curve should be a parabola since the centrifugal force varies as the square of the speed. A parabola was accordingly fitted to the data by the method of least squares, resulting in the following formula:

$$R = 0.22\ V^2$$

where R is the radius of curvature in feet and V is the speed in miles per hour. The comfortable rate of deflection as computed from this formula is 9.7 feet per second per second, and the tendency to skid is 0.3 the weight of the vehicle, which would be generally accepted as a safe condition. Computed comfortable speeds for various radii of curvature are given in Table 1.

---

[a] Digested from a thesis prepared for the masters' degree at Albert Russel Erskine Bureau for Street Traffic Research, Harvard University.

It is obvious that higher speeds may be maintained with safety, and, if the curve is followed for only a short distance, without particular discomfort.

TABLE 1.—*Comfortable driving speeds for various radii of curvature*

Radius	Speed	Radius	Speed	Radius	Speed
Feet	*Miles per hour*	*Feet*	*Miles per hour*	*Feet*	*Miles per hour*
20	10	200	30	600	52
30	12	250	34	700	56
50	15	300	37	800	60
100	21	400	43		
150	26	500	48		

Splayed intersections.—Closely related to the improvement of curb radii is the practice of splaying or widening the roadways at the intersection. This device has been used in many places. Successful examples are found in Cook County, Ill. and in Reno County, Kans. The Illinois design provides an additional lane on either side of the pavement for a distance of 300 feet from the intersection. The Kansas design eliminates the intersection altogether for turning traffic by providing turning roads of 200 foot radius. Such treatment reduces congestion by removing much or all of the turning movement from the intersection area. The design of an intersection of this type depends upon the locality, the available right of way, the volume and character of traffic, and the percentage of turning movement. A suggested design is shown in Figure 2.

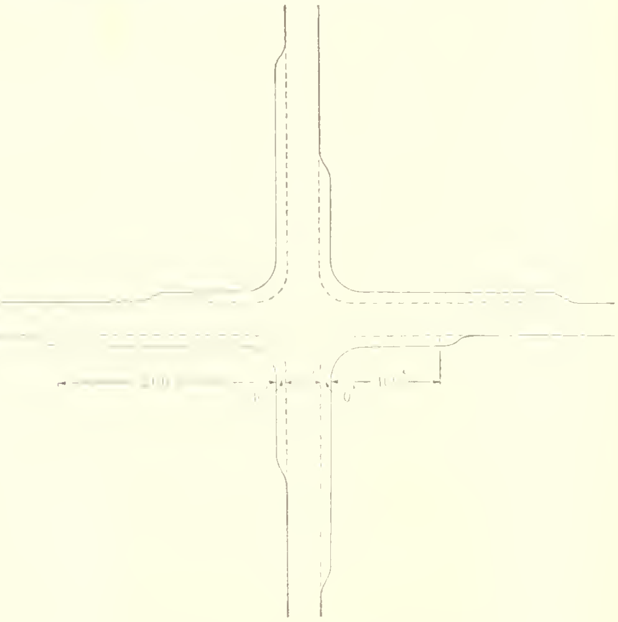


FIGURE 2.—INTERSECTION OF TWO 30-FOOT ROADWAYS SPLAYED FOR TURNING

At heavily traveled intersections where control methods result in considerable stoppage, splaying may prove advantageous to through traffic as well as to turning traffic, by permitting the accumulation of vehicles in a greater number of lanes, with a resultant shortening of the waiting lines. Before splaying an intersection for this purpose, care must be taken that the greater number of lanes may be accommodated beyond the intersection to such a point that the vehicles may resume their travel in the normal number of lanes. Each additional lane of width increases the pedestrian hazard, and necessitates a longer clearance period in the control device.

Islands.—To facilitate movement and to prevent confusion, islands may be installed within the intersection or in the entering streets. This is especially helpful when splaying or wide streets result in an intersection of extensive area.

Medial strips are useful where it is desirable to separate opposing traffic movements and at the intersection of wide routes where proper alignment on either side of the cross streets is difficult. The width should be at least 4 feet and should be varied to such an extent that the area remaining on either side can be divided into 10-foot traffic lanes, with the possibility of one 7 or 8 foot parking lane. The construction may be concrete or a curb-inclosed grassed or planted area. In any case, the curb height should be from 8 to 10 inches and the end confronting traffic amply protected at night by a flashing or reflecting beacon.

Directional islands are normally located in the throats of the streets composing the intersection and their size and shape must be very carefully considered in order that they will assist rather than hinder the flow of traffic through the intersection. They are normally placed in a dead area and are shaped according to the desired flow of traffic. The islands should extend into the throats of the entering streets for such a distance that there can be no mistake as to the route to be followed and, in case the islands are to be used for pedestrian safety zones, they must extend to a point beyond the crosswalk and be amply flood-lighted.

The natural tendency for a driver is to continue in a straight line unless compelled to change his route because of external conditions. Therefore, it is imperative that these islands, intended to align traffic properly, be so designed that the obvious route for a driver to follow as he passes the island is the route which will bring about the most effective operation of the intersection.

Directional islands may often be located nearer the center of the intersection to delimit a neutral area which otherwise would be a point of conflict for several lanes of traffic. The sides of islands of this sort which vehicles are expected to follow in turning must have a radius which fulfills the requirements previously stated under curb radii.

Many times directional islands are located at the center of the intersection to serve merely as a point about which traffic turning left must proceed. In this case, it is not necessary to have a minimum radius as specified above, for the vehicle does not follow the island for any appreciable distance. The island here merely serves to indicate to the driver the route which will least interfere with the opposing traffic. Its location must be such that any vehicle turning about it, presumably to the left, will be able to do so at a convenient turning radius and will not be required to swing out either before or after passing it with consequent fouling of other lines of traffic. It will be noted that the ordinary flashing beacons so often seen at the center of intersections fall under this classification and their location should be determined in the above manner. Applications of this theory are shown in the designs which accompany this article.

ROTARY TRAFFIC

Rotary traffic is here defined as the movement of vehicles around a traffic circle or square in one direction only. Such traffic circles and squares are usually formed by the convergence of several streets. Where the rotary system is in operation, vehicles are required to execute a right turn upon entering the circle, proceed around the circle to the right, interweaving with the traffic already upon the circular roadway and leave at the desired exit by means of a right turn. The circle should be so designed that no vehicle can pass directly through the intersection. In effect this plan creates a circular 1-way street which can be entered or left only by means of a right turn, no left turns being permitted. The rotary traffic plan intends a non-stop flow of traffic although in special cases it has been found necessary to control traffic by officers or lights for the convenience of pedestrians.

The rotary movement of traffic is graphically pictured in Figure 3, a typical design for the right-angled intersection of two 4-lane streets or highways, the paths of the vehicles being indicated by means of arrows. It will be observed that there are four points of conflict, one such point arising about halfway between any two entering streets. The basic theory of the system presupposes that actual conflict will be reduced to a minimum since entering vehicles are forced by the form of the intersection to interweave with the traffic already upon the circular roadway by proceeding in the same direction. The two traffic streams converge and mingle and there is no direct conflict of direction. The flatter the angle of convergence the easier and more efficient is the interweaving process. This angle of convergence bears an inverse relationship to the diameter of the central island, the angle decreasing as the diameter is increased. However, there are factors which limit the size of the central island, such as the amount of land available and the greater inconvenience to the traffic as the length of travel about the circle is increased.

The central island.—The function of the central island is to produce a true rotary movement. For best results, conflicting streams of traffic should converge at an oblique angle and there should be sufficient length of roadway between entering streets for a complete and easy interweaving of vehicles. The shape and size of the central island or obstruction around which traffic is required to pass, therefore, has a marked influence on the efficiency with which traffic is handled. Ordinarily the circular form of central island is desirable as best fulfilling the above conditions, but for intersections of radical design special forms of central island will be required. At a right-angled intersection where the traffic is much heavier on one street than on the other it is usually desirable to give traffic on the major road preference by using an elliptical form of central island, the long axis of the ellipse coinciding with the axis of the major route. By this method the traffic on the major route is given the advantage of easy curves at the intersection and will be able to pass through with greater speed and less delay than traffic crossing the major route. Cross traffic will be forced to slow down considerably and will filter through the main traffic or turn into the main traffic stream when opportunity offers with a minimum of delay to the major traffic.

The size of the central island is a function of the convenient turning radius of vehicles and the length between the entering streets required for the interweaving of vehicles. Where traffic volume is large, this distance should be at least 100 feet for efficient operation although satisfactory results have been obtained where the interweaving distance is considerably less. It has been observed, however, that where less than 100 feet is provided, the speed of movement is considerably reduced as the traffic increases in volume.

The size of the central island from the point of view of the speed which may be maintained through the circle may be determined from Table 1. As mentioned before, this table is conservative and the speeds will vary greatly with the individual driver's idea as to what constitutes a comfortable turning speed; but it does serve as an indication of the possibilities of a design of this sort.

The radius of the central island is not the only factor which controls the speed of vehicles through the rotary intersection. It would appear from Table 1 that a central island with a radius of 400 feet would permit vehicles to pass through at a speed of approximately 40 miles per hour. Such would not be the case in practice. Vehicles would necessarily be retarded by the interweaving of conflicting streams of traffic. Speed studies on several rotary intersections indicate that average speed of vehicles in passing through the intersection is between 15 and 20 miles per hour.

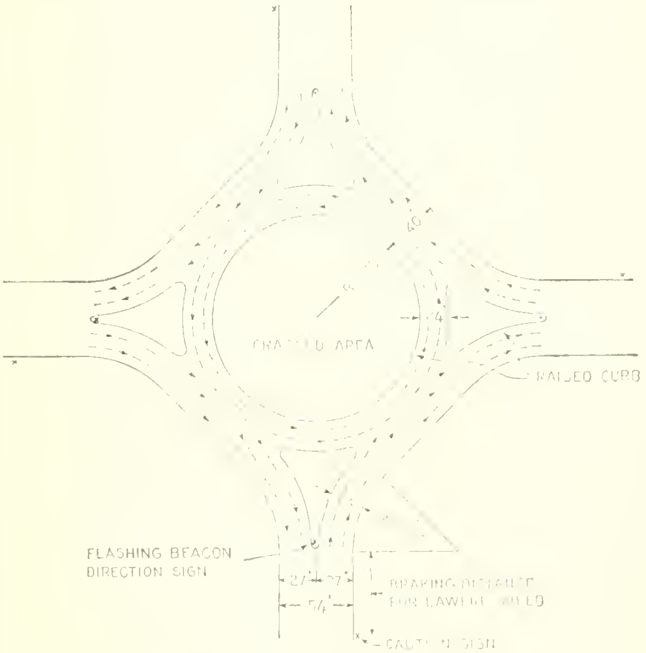


Figure 3.—Recommended Design for Rotary Intersection with Central Island of Minimum Diameter

The traffic studies at Lee Circle, New Orleans, show that during the maximum hour 2,543 vehicles passed through the circle and that the average speed during that time was between 16 and 19 miles per hour. Several observations at the rotary traffic intersection of Old Colony Avenue and Columbia Road, Boston, indicate that the average speed through this circle is between 15 and 20 miles per hour. While the radius of the central island at Lee Circle is 140 feet and the radius at Old Colony Avenue and Columbia Road is only 88 feet, the average speeds of vehicles passing through the intersections are remarkably similar.

Observations at these two intersections and others indicate that an average speed of 15 to 20 miles per hour is safe and that interweaving of traffic can be accomplished at this speed satisfactorily. It is believed that speeds much in excess of 20 miles per hour are dangerous except where the intersection has been designed along the elliptical form to pass traffic upon a major route with a minimum of delay, and where cross traffic is light and turning movement negligible.

Observations of actual intersections indicate that central islands should not be designed with radii of less than 75 feet. At the customary right-angled intersection of two 40-foot roadways, a central island with a radius of 75 feet will allow vehicles to pass around the island comfortably at a speed of about 20 miles per hour and will also give a satisfactory distance between the entering streets for the interweaving of traffic.

Width of rotary driveway.—In an effort to determine a logical theory of design for the width of the rotary driveway, the following formula has been evolved as a basis from which to work. A symmetrical case is assumed, as shown in Figure 4 and all traffic is assumed to pass directly through the circle, making no right or left turns. It is also assumed that traffic streams occupy the same width upon the circular roadway as upon the entering streets. This is believed to be the case in practice. Observations show that there is little or no tendency for vehicles to cut out and pass each other while upon the rotary driveway and it has also been shown that there is little increase in the capacity of a lane of traffic, measured in vehicles per hour, as the speed of vehicles increases above 15 miles per hour. In other words, while a stream of traffic approaching a rotary intersection might be slowed from an average speed of 30 miles per hour to an average speed of 15 miles per hour, the capacity of such a lane would not be materially decreased and there would be no tendency for the traffic stream to become broader.

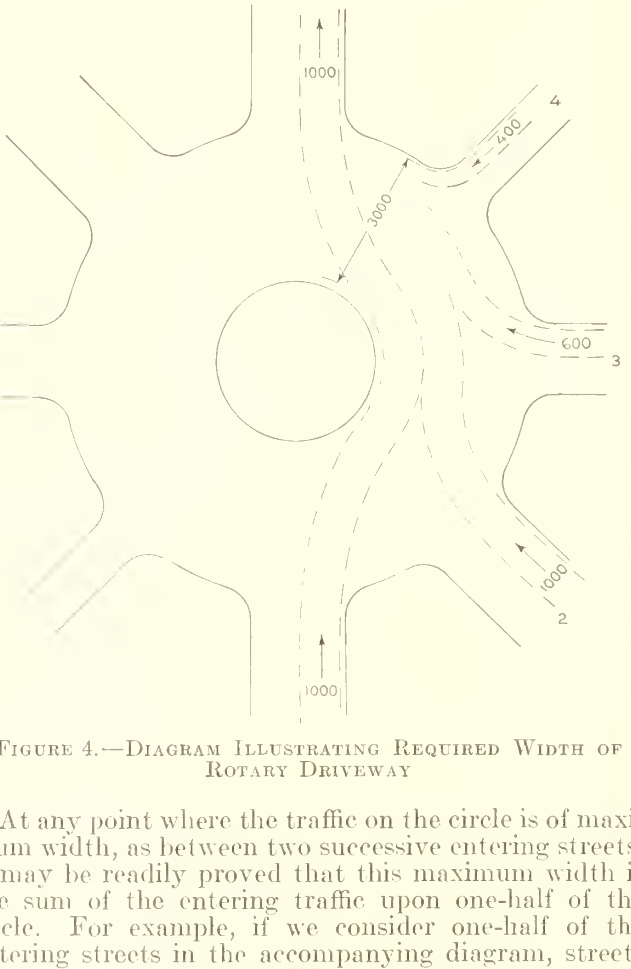


FIGURE 4.—DIAGRAM ILLUSTRATING REQUIRED WIDTH OF ROTARY DRIVEWAY

At any point where the traffic on the circle is of maximum width, as between two successive entering streets, it may be readily proved that this maximum width is the sum of the entering traffic upon one-half of the circle. For example, if we consider one-half of the entering streets in the accompanying diagram, streets Nos. 1, 2, 3, and 4, the width of the traffic stream on the circle just past street No. 4 will be 1,000 + 1,000 + 600 + 400 = 3,000 cars per hour (here the term "width of traffic stream" is understood to mean the graphic width of the band representing traffic density). If we let t designate traffic density upon the entering streets and T the traffic density upon the circle, the density of the stream at a maximum point upon the circle may be expressed as follows:

$$T=\frac{t_1+t_2+t_3+\text{--------}t_n}{2}$$

As the ultimate capacity of any street is in general determined by the number of traffic lanes upon that street, it seems desirable to express the width of the circular roadway in terms of the total number of traffic lanes on the streets entering the circle. If both entering and leaving lanes of traffic be included, the total must be divided by 4 instead of 2 to hold the same relation, as there are as many lanes entering as leaving the circle. The width of the circular roadway may then be expressed in terms of the total number of traffic lanes as follows:

$$\begin{array}{c}\text{Number of traffic lanes}\\ \text{of circular driveway}\end{array}=\frac{\begin{array}{c}\text{Sum of all traffic lanes}\\ \text{of entering streets}\end{array}}{4}$$

Any such formula must necessarily be modified in practice. In any case, it seems desirable to have at least two traffic lanes upon the circular driveway, one for cars passing through and one for cars making right turns. It is also doubtful if more than six lanes are desirable. Large open spaces tend to confuse drivers and the traffic pattern tends to lose an orderly channelized form. Widths as great as six traffic lanes require lane markers, which are objectionable upon the rotary driveway.

Curb radii.—Curb radii should in all cases correspond as nearly as possible to the radius of the central island. It has been observed that where curb radii are small, traffic tends to make the turn upon entering the circle as broad as is convenient, with the result that vehicles are carried away from the outer edge of the circular drive, and a large percentage of unused pavement results. In Figure 8, F is shown a trafficp attern of the Old Colony Avenue-Columbia Road Circle in Boston. This pattern clearly shows the effect of short curb radii. In this case, approximately 20 per cent of the total area of the circular driveway was never used by traffic.

At this point it may also be pointed out that reverse curves made by curb radii and the outer edge of the circular drive are ignored by traffic. It is much better to follow this traffic tendency in design and to eliminate such reverse curves by making the outer edge of the drive in effect a square or a polygon, as shown in Figure 3.

Directional islands.—Directional islands here refer to obstructions in the throats of the streets entering a traffic circle, the principal purpose of which is to channelize the entering traffic and to direct it into the rotary traffic stream at an oblique angle. The shape of the island should be such that the entering traffic is properly guided in its movement and that no vehicle can break out of line to take an undesired course into the rotor. This effect is best attained by the use of triangular islands, which are naturally formed by the traffic pattern, the base of the island being an arc concentric with the central island and the apex extending well up into the entering street. The photograph of Scott Circle, Washington, D. C. (fig. 5), shows very clearly the dead space at the entrances of the converging streets which traffic does not use and which should determine the shape and position of the islands.

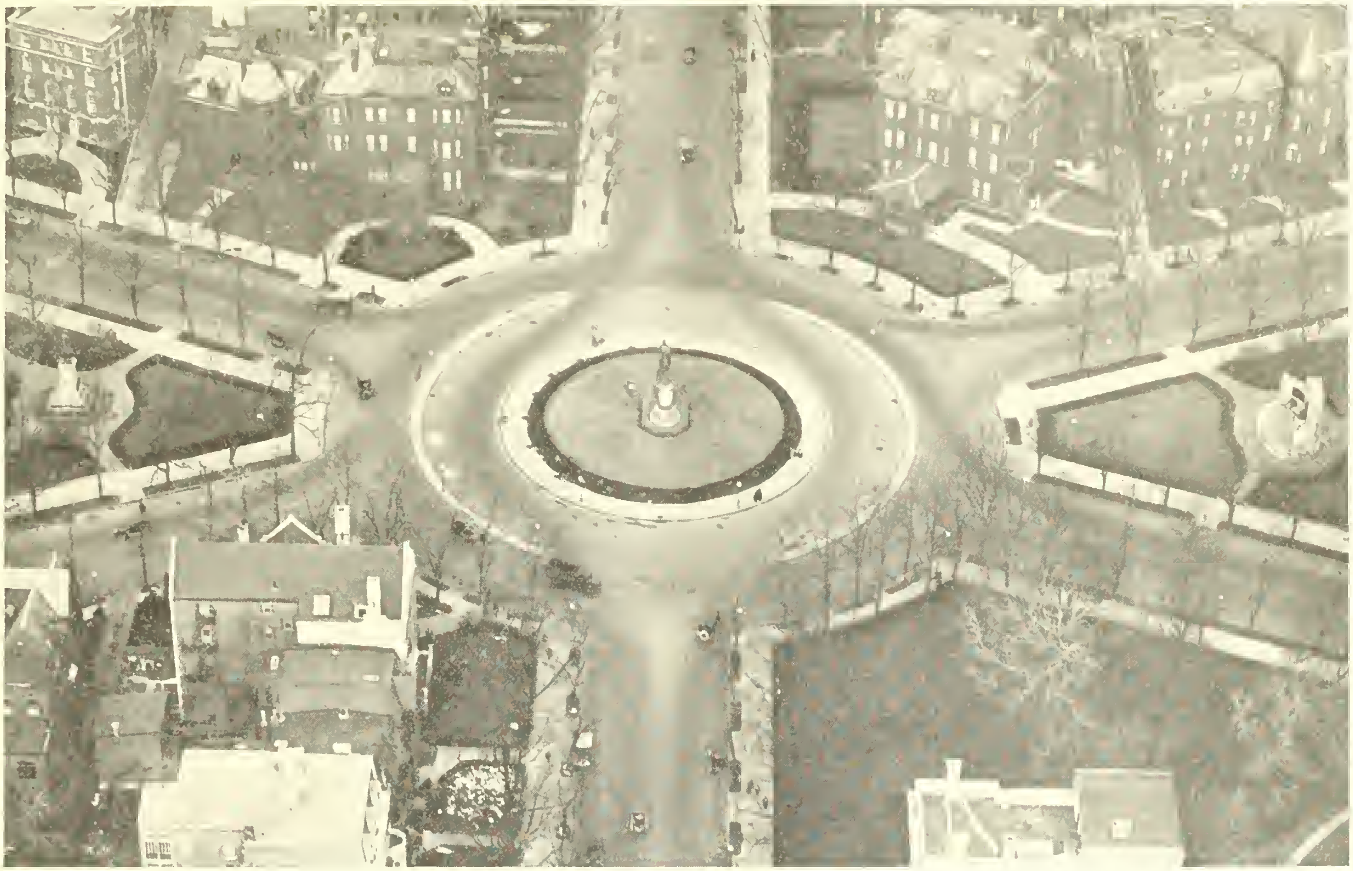


Figure 5.—View of Scott Circle, Washington, D. C., Showing Areas Unused by Traffic at Entrances of Converging Streets

In designing the sides of the islands it is important to bear in mind the basic principle that in so far as is possible the routes followed should be the natural routes of the traffic. The sides should, therefore, be arcs concentric with the curbs of the entering streets and so terminated that traffic entering the circle is released parallel to a tangent to the central island and traffic leaving the circle is not deflected from a straight line course, paralleling a tangent to the central island.

The width of pavement necessary between the base of any one of the directional islands and the central island should not be greater than that required to provide for the expected volume of traffic at that point. In the case of a right-angled intersection of two 4-lane streets or highways carrying an equal volume of traffic, the necessary width is two traffic lanes. In this case, where traffic negotiates a curve of comparatively small radius, it is recommended that 12-foot lanes be provided.

For the sake of appearance and permanency, the directional islands should be constructed of concrete or masonry curbing, inclosing a graveled or sodded area. Where the location of the rotary intersection is such that pedestrian traffic must be provided for, the directional islands will also function as pedestrian safety islands and should be so constructed. The surface of the island should be given a rough finish to insure safe footing and the end of the island farthest away from the intersection should be provided with a concrete buttress or post to protect pedestrians from reckless motorists. The buttress will also serve to support a directional sign.

Signs.—In addition to the directional islands, traffic should be guided in its movement by appropriate signs. Such signs should serve two purposes: First, as a warning to approaching vehicles of the rotor which they are about to enter and the necessity of proceeding with due caution; and, second, as a means of directing traffic movement on the rotor. In existing circles this purpose is usually accomplished by a series of warning signs upon the entering streets and 1-way arrows at appropriate points upon the circle. A logical system of signing is indicated in Figure 3.

Minimum and maximum designs.—In order to illustrate the application of the theories developed in the preceding discussion, two typical designs have been prepared, one suggested as the minimum desirable installation, while the other is intended as an example of the maximum size beyond which no improvement in operation may be gained.

As the rotary installation requires considerably more area than the ordinary intersection, the cost of the extra land will limit the size of the circle at many locations. The design decided upon will be the result of balancing the cost of land against the size necessary for efficient operation.

The design shown in Figure 3 is suggested as the minimum desirable circle for the intersection of two 4-lane highways or streets and should be applied at a location where land values are relatively high and traffic speeds are moderate.

The basic requirement which should determine the minimum size is the distance necessary for easy interweaving of the vehicles and the angles at which converging lanes approach each other. As previously noted, the interweaving distance should not be less than 100 feet for good operation and the angle of convergence should not exceed 30°. In the present design, these requirements are satisfied by a central island 150 feet in diameter as the distance between any two consecutive directional islands is 100 feet and the angle between any two conflicting lanes of traffic is less than 30°.

A central island 150 feet in diameter gives a turning radius of 75 feet and satisfies the minimum conditions for convenient operation, permitting a speed of about 20 miles per hour around the circle.

It will be noted that the curb radii are also 75 feet to correspond with the radius of the central island and that the outer curbs of the circular roadway are straight sections joining the curbs of the entering streets. This facilitates the right turning movement, for the same speed may be maintained when turning right as when proceeding through the circle. There is no tendency to swing out into the traffic upon the circle and all the pavement area is utilized.

The islands in the throats of the entering streets occupy the area which would normally not be used by traffic and are so shaped that they channelize the traffic movement, but in no way interfere with the natural routes. The sides of the islands are circular arcs, 24 feet from and concentric with the curbs of the entering streets, while the base in each case is an arc concentric with the central island and 24 feet from it. This allows for two 12-foot lanes on all sides of the island, the lane width recommended for curves of comparatively small radius. The sides of the islands are so shaped that traffic is released on a tangent to the circle and appropriate warning and directions are provided. As shown in the diagram, the roadway is designed to carry two lanes of traffic on all parts of the circle with provision for four lanes between entering streets.

The diagram of the possible traffic paths would indicate 16 possible conflict points. A condition such as this would probably never exist, even in the heaviest traffic. For illustration, consider the case of a driver going straight through the circle. When there were two lanes moving on the circle, his tendency would be to remain in the outer lane and to head directly for the exit street on passing the central island. If, however, he wished to make a left turn, following around the rotor for 270°, he would in all probability enter the outer lane of traffic and at the first opportunity move into the inner lane next to the rotor, remaining there until he approached the street on which he intended to leave the circle. When an opportunity offered, he would mingle with the vehicles in the outer lane and leave on a tangent as did the first driver.

If the drivers acted as outlined above, which is a reasonable assumption, they would cut only the two lanes of traffic upon the circle and this reduces the number of conflict points to a possible maximum of eight, or one for each lane of traffic. There will ordinarily be but a single line of traffic around the rotor and probably one on each entering street, thus reducing the number of conflict points to four.

Since land values and costs fluctuate widely, there has been no attempt to estimate the cost of constructing the traffic circle illustrated in Figure 3. However, the following comparison of the rotary design with an ordinary intersection is made, 100-foot rights of way being assumed for each of the highways:

Item	Rotary	Ordinary intersection	Excess
	Square yards	*Square yards*	*Square yards*
Grading	6,400	3,600	2,800
Paving	3,716	3,600	116
Island area	2,697	None.	2,697

Approximately 1,800 square yards of land will be required in addition to that included in 100-foot rights of way.

In many cases, where land is less expensive and the acquisition of extra land at the intersection is not an important consideration, the size of the circle may be increased to good advantage. In the design suggested as a minimum, the entering lines of traffic intersect those leaving at the next street at an angle of approximately 30°. As the diameter of the rotor and the curb radii of the entering streets are increased, this angle becomes smaller until at some point the entering and leaving lanes become tangent to the rotary traffic lanes, as shown in Figure 6.

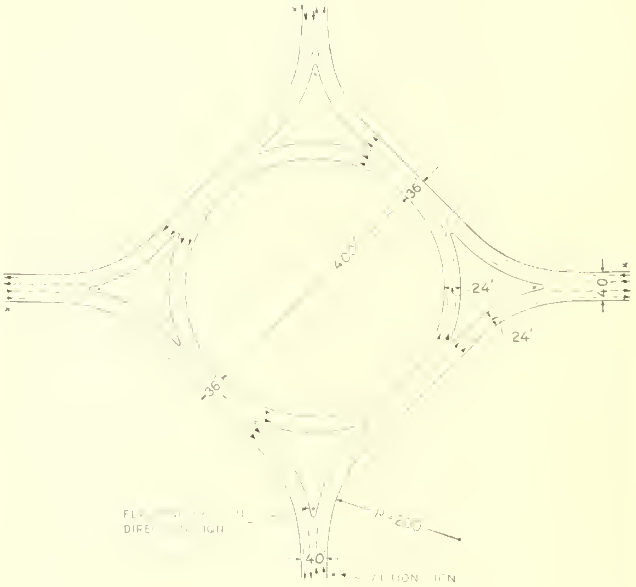


Figure 6.—Recommended Design for a Rotary Intersection with Central Island of Maximum Diameter

It is obvious that any further increase in the diameter of the circle will not result in a corresponding improvement in operation. The entering lanes of one street merely become tangent to the rotary movement before the exit lanes of the next successive street leave it and, in addition, it becomes necessary to introduce the objectionable reverse curve in the outer curb.

The physical features of this circle are designed in accordance with the general principles followed in the design of the minimum circle. The central island and the curbs of the entering streets are of equal radius and the straight outer edge of the circular roadway eliminates the reverse curve in turning right. The islands are designed according to the traffic pattern and since the distance between islands is much greater than is the case of the smaller circle, the roadway width may be less and still permit easy interweaving.

From the standpoint of efficient operation, this circle is the largest that should be used. There is no conflict between entering and leaving lanes and with reasonably skillful driving there should be no conflict of traffic lanes upon the circle. The only reduction of speed is occasioned by vehicles interweaving and jockeying for position. Table 1 indicates that speeds of 30 miles per hour may be maintained on the circular roadway with comfort. A circle of this sort would be rarely installed, but it may be advantageously used at the intersection of two heavily traveled rural highways or at such locations as parkways, civic centers, and the like.

The land required in excess of the 100-foot right of way is approximately 14,500 square yards, and the quantities are as follows:

Item	Circle	Ordinary intersection	Excess
	Square yards	*Square yards*	*Square yards*
Grading	27,800	7,000	20,800
Paving	9,800	7,000	2,800
Island area	18,000	None.	18,000
Curbing	[1] 5,740	[1] 3,120	[1] 2,620

[1] Linear feet

Adaptability of rotary traffic.—In conclusion, it may be well to point out the advantages of the rotary traffic system and to suggest the limiting conditions which will determine the expediency of its application at a particular location. The greatest apparent advantage of rotary traffic is that in theory it presupposes a continuous movement of traffic, direct conflict of vehicles and the troublesome left turn being eliminated by the device of compelling all traffic to proceed in one direction around a circular drive. The system is capable, therefore, of handling a large volume of traffic with little delay.

Another very distinct advantage is the reduction in accident loss. The shape of the true rotary intersection is such that traffic is compelled to slow down and the paths of vehicles do not cross at right angles, but rather converge and interweave. Any accident which occurs, therefore, is the result of a small differential of speed, a following car colliding with a slower vehicle proceeding in the same direction, or side-swiping a vehicle during the interweaving process. Head-on collision, or a right-angled conflict of vehicles, is virtually impossible.

That safety is improved by rotary traffic is evidenced by Table 2, which gives the experience in Los Angeles, Calif., before and after the installation of a traffic circle.

In spite of these obvious advantages there are several disadvantages which seriously restrict the application of rotary traffic in the ideal symmetrical form. The cost of the excess land required is a significant factor where land values are high. It would be economically unsound, for example, to install even the minimum suggested design at intersections in the business district of a city. The cost of excess land at such a location would be so great that it would probably be more economical to construct a grade separation.

A second factor is that of topography. The rotary system is designed for operation on level circles and any attempt to install a rotary intersection on a grade would result in obvious complications. As far as is known, there are no installations where the circle itself is on a grade.

TABLE 2.—*Number of accidents and property damage before and after the installation of a traffic circle in Los Angeles, Calif.*

Intersection at Wilshire Boulevard and Western Avenue			Neighboring right-angled intersection at Wilshire Boulevard and Vermont Avenue	
1922	Accidents	Property damage	Accidents	Property damage
4 months, before installation of traffic circle	50	$1,491	30	$833
6 months, after installation of traffic circle	32	353	62	1,640

A most important consideration is the character of the traffic. The system operates to greatest advantage when traffic is uniform in character, as a variety in type of traffic seriously reduces its efficiency. It was intended primarily to facilitate motor-vehicle traffic of roughly uniform operating qualities. When, in addition, pedestrian and street car traffic must be provided for, it is necessary to modify the system to a considerable extent.

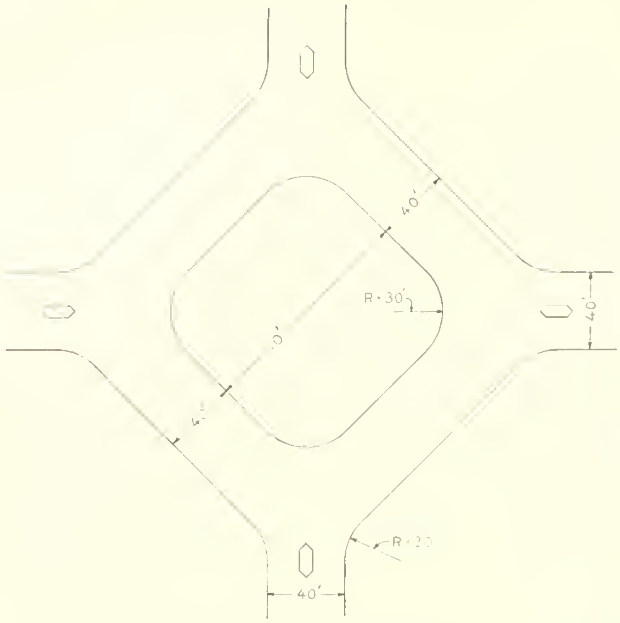
In the usual rotary intersection, vehicular flow is uninterrupted and the pedestrian is at a decided disadvantage. The situation, however, is similar to that at any uncontrolled intersection and unless there is a very large volume of traffic, no particular provision need be made for pedestrians. The auxiliary directional islands serve as convenient refuges and it is suggested that pedestrian crosswalks be laid out to utilize the safety areas thus provided. By using the islands in this manner, the pedestrian crosses no more than two or three lanes of traffic at one time, and encounters vehicles moving in one direction only.

When vehicular traffic is very heavy and when, in addition, there is a large volume of pedestrian traffic, it will be necessary to provide pedestrian subways or to interrupt the vehicular flow occasionally for the convenience of pedestrians. This is easily accomplished by the installation of traffic signals as at an ordinary intersection.

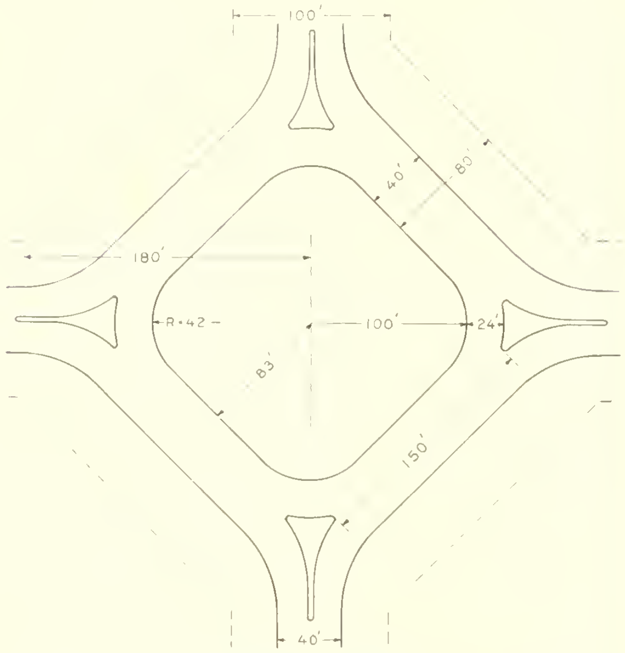
Street cars present a more difficult problem. In a great many instances where the rotary system has been applied to regulate traffic on an existing circle or square, tracks are so located that street cars conflict with the rotary movement of vehicles, the street cars generally proceeding directly across the circle. In periods of light traffic this conflict is not serious, but as the traffic volume increases it becomes necessary to interrupt movement upon the rotary roadway for the passage of street cars.

Square or polygon-shaped central islands.—The use of square or polygon-shaped central islands in the design of rotary intersections reduces the efficiency of the system and any advantages which may be attributed to this type of design are enhanced in the circular design. This particularly is true where the minimum size of rotor is contemplated for a given intersection.

Figure 7 illustrates two types of rotary intersection in which a square central island is used, the island being so designed that the sides of the square are at an angle of 45° to the center lines of the streets forming the intersection. The intersection in each case is a simple right-angled one and the purpose of the designers has evidently been to evolve a rotor requiring a minimum amount of land compatible with efficient operation.



BRITISH ROUNDABOUT DESIGN



COOK COUNTY DESIGN

Figure 7.—Rotary Intersection Designs with Square Central Island

The British design appears in A Report on the Layout of Crossroads, Junctions, and Corners, submitted by the British Ministry of Transport to all road authorities in England. This report was published in full in the The American City for November, 1929, Volume XLI, No. 5, pages 151–154. The design which appears here was recommended for use at the crossing of two roads of equal traffic importance. The Cook County design is included in the report of the Cook County, Ill., Grade Separation Advisory Committee, August 5, 1929.

The greatest apparent advantage of this type of design is that it permits better arrangement of the building lots about the intersection than does the circular form. However, if the outer curbs of the intersection are designed as straight lines, as suggested in Figure 3, rather than as arcs concentric with the circular island, this apparent advantage disappears. There remains only the possible advantage to be gained in a saving of land required by the square design.

Table 3 gives a comparison of the dimensions of the two square designs and the recommended circular design, showing the various areas of pavement and islands required:

Table 3.—*Dimensions of central island and land areas required for British and Cook County designs, as compared with recommended circular design of Figure 3*

Design	Dimensions of central island	Area pavement		Area islands		Area excess land taken	
		Square yards	Per cent[1]	Square yards	Per cent [1]	Square yards	Per cent [2]
British	110 feet square	4, 157	112	1, 281	47. 4	860	47
Cook County	166 feet square	4, 224	113	2, 982	110. 5	2, 300	126
Recommended	150 feet diameter circle	3, 716	100	2, 697	100. 0	1, 820	100

[1] Percentage of recommended design.
[2] Excess area required to be taken based on 100-foot right of way in each case.

In spite of the slight advantage in the amount of land required in the case of the British design, there appear to be several serious objections to the square design from the point of view of traffic efficiency. These objections may be made clear by a detailed discussion of the designs in Figure 7.

The first objection is the necessity of using a very short radius in rounding off the corners of the square island. In the British design, while the square is 110 feet on a side and gives an admirable distance between entering streets for interweaving, the curves rounding off the square have a radius of only 30 feet. This radius is but slightly larger than the minimum turning radius of our longer-wheelbase automobiles. It is obvious that in negotiating this rotor traffic will be compelled to travel very slowly to remain in the channels provided for it or it will maintain a normal rate of speed and avoid the designed channels. At a speed of 15 or 20 miles an hour, the vehicles occupying the inner lane of traffic next to the central island will be thrown toward the outer lane on rounding the corners, with resulting conflicts and possible accidents. The traffic proceeding about the square will tend to take a circular course and the traffic pattern will undoubtedly show the square inscribed in a rough circle. The 30-foot curb radii of the entering streets will bring about a similar effect along the outside of the rotor when vehicles make a right turn. These two tendencies reduce the effective width of the roadway and concentrate traffic in the two center lanes.

The Chicago plan has some advantages over the English design. The curb radii are 75 feet, and cars turning to the right with the intention of leaving by means of the next street can negotiate the turn at about 20 miles per hour without swinging wide into the inner lane. However, the design is not consistent in that the radius of the curves rounding off the square rotor is only 42 feet. While a right turn can be made at a speed of 20 miles per hour, a car proceeding directly through the intersection must slow down considerably in making the 42-foot radius curves. If speed is not so reduced, the motorist will be forced to swing out into the next traffic lane and the same disadvantages hold as for the English design.

EXISTING TRAFFIC CIRCLES DISCUSSED

In an attempt to gather information concerning the design and operation of existing traffic circles, a questionnaire was sent to all cities with a population of 100,000 or more and to other cities in which it was known that traffic circles were in operation. A summary of the data assembled through the courtesy of the various city engineers and members of city planning commissions appears in Table 4. This table, while containing all the information received, is admittedly incomplete, but will reveal particularly the wide variance in practice in the design of circles. In view of this fact, several plans are presented, each of which has one or more particularly good features which, if incorporated in a single design, would result in an intersection almost coincident with that presented in the previous chapter.

It will be seen in the table that the speed of vehicles passing through the intersection was given in only two cases. Despite the wide difference in both the diameter of the rotor and the volume of traffic, the speed maintained is practically the same. It is believed that this speed will be relatively constant for all circles, but any additional information both as to volume and speed which may be maintained upon existing circles would be very valuable to one considering the installation of such a design.

Monument Circle.—Monument Circle[1] in Indianapolis, Ind. (fig. 8, A), represents good practice in existing traffic circles. It was originally planned in 1821 by Ralston, an assistant to Major L'Enfant, as the center of a city 1 mile square with four broad avenues radiating from it. At the present time, the circle passes 1,500 to 2,000 vehicles per hour without interruption and without regulation by traffic officers or signal-control lights. It is also the terminal of all city bus lines.

Features of note are the large radius of the central island and the design of the islands in the entering streets. These islands were installed in response to traffic demands and for pedestrian safety. It is worthy of note that while the areas are merely outlined with stanchions and traffic buttons, the operation is efficient.

The circular driveway width, which is entirely adequate in this case, is the same as that given by the formula previously suggested, that is, one-quarter of the width of the entering streets. The operation of the circle would undoubtedly be improved by increasing the curb radii of the entering streets, but there is serious question if increased efficiency would compensate for the expense of such a proceeding in the heart of the city.

Niagara Square.—Niagara Square, Buffalo, N. Y. (fig. 8, B), is a good example of the rotary system applied to a large square. The corners of the central island have been cut back to such an extent that it is practically a circle and large turning radii are provided for traffic circling the island. It will also be noted that the curbs of the entering streets are cut back to provide a turning radius sufficient to allow entering vehicles to execute the right turn without interfering with the traffic already upon the square.

As there are seven entering streets, the interweaving distance in several places is less than has been recommended; but because the heavy traffic flow is largely confined to Niagara Street and Delaware Avenue, the operation of traffic through the square is not seriously affected. Here the width of the circular roadway is less than the maximum given by the formula and this width is still further reduced by vehicles parked around the outer curb, illustrating that the formula gives a very generous roadway width and that traffic can be handled efficiently with less width.

TABLE 4.—*Existing traffic circles in cities of 100,000 or more population in the United States*

Circle	Location	Diameter of circle	Total width of entering streets	Width of circular roadway	Number of entering streets	Maximum vehicles per hour	Speed in miles per hour
		Feet	*Feet*	*Feet*			
Lee	New Orleans, La.	280	202	40	4	3, 400	16–19
Claiborne	do	210	268	31	7	......	......
Druid Hill Park	Baltimore, Md.	115	236	85	7		......
Niagara	Buffalo, N. Y.	305	374	75	7	3, 500	......
Monument	Indianapolis, Ind.	333	200	50	4	2, 000	......
Du Pont	Washington, D. C.	386	464	40	10	2, 176	......
Sheridan	do	190	164	40	3	......	......
Thomas	do	208	410	50	8	3, 300	......
Scott	do	120	450	60	6	2, 350	......
Iowa	do	380	324	36	8	2, 500	......
Washington	do	350	360	40	8	......	......
Grant	do	300	300	40	8	......	......
Sherman	do	320	300	50	8	......	......
Lincoln Memorial	do	640	290	60	6	......	......
Chevy Chase	do	240	350	70	8	......	......
Laurelhurst	Portland, Oreg.	190	192	8	4	......	......
Central Park	do	240	264	36	8	......	......
Colorado and Orange.	Pasadena, Calif.	60	194	70	4	......	......
Columbia Road and Old Colony Avenue.	Boston, Mass.	175	177	60	4	3, 000	15–20
Kaign Avenue	Near Camden, N. J.	320	310	50	6	3, 200	......
Whitehorse Pike	do	[1] 110	145	45	5	2, 465	......
Brooklawn East	Near Brooklawn, N. J.	200	167	36	4	......	......
Brooklawn, West	do	155	135	33	3	......	......
Ridge Road Bridge approach.	Rochester, N. Y.	216	300	60	6	......	......
Lindell (proposed)	St. Louis, Mo.	130	320	70	5	......	......

[1] Minimum.

A traffic count made on July 7, 1929, showed that 3,548 vehicles entered the square during the maximum hour and passed through without delay or confusion.[2] The advantages of the design are the large radii of the central island and the curbs of the entering streets.

Circles in Portland, Oreg.—Two traffic circles of Portland, Oreg., present examples of good design. Figure 8, C is the intersection of two main traffic streets in a residential district. Both carry a large volume of traffic which passes through the circle without interruption.

An interesting feature of this design is the exceptionally large radii of the curbs of the entering streets. A radius of 200 feet gives a continuous curve for traffic turning right from one entering street into the next and results in the outer curb of the circle being convex with respect to the central island. Where land is not exceptionally expensive, this practice offers some advantage over the straight outer curb design. Very little roadway width is wasted and vehicles are encouraged to follow a natural route. The radius of the central island is 95 feet, resulting in a satisfactory interweaving distance.

Figure 8, D illustrates a treatment where a number of streets converge at a point. The radius of the central island is too small to give a satisfactory interweaving distance provided all the streets were carrying a heavy traffic load, but it is effective in this case because the traffic is concentrated on Ladd Avenue and the other streets are of comparatively minor importance.

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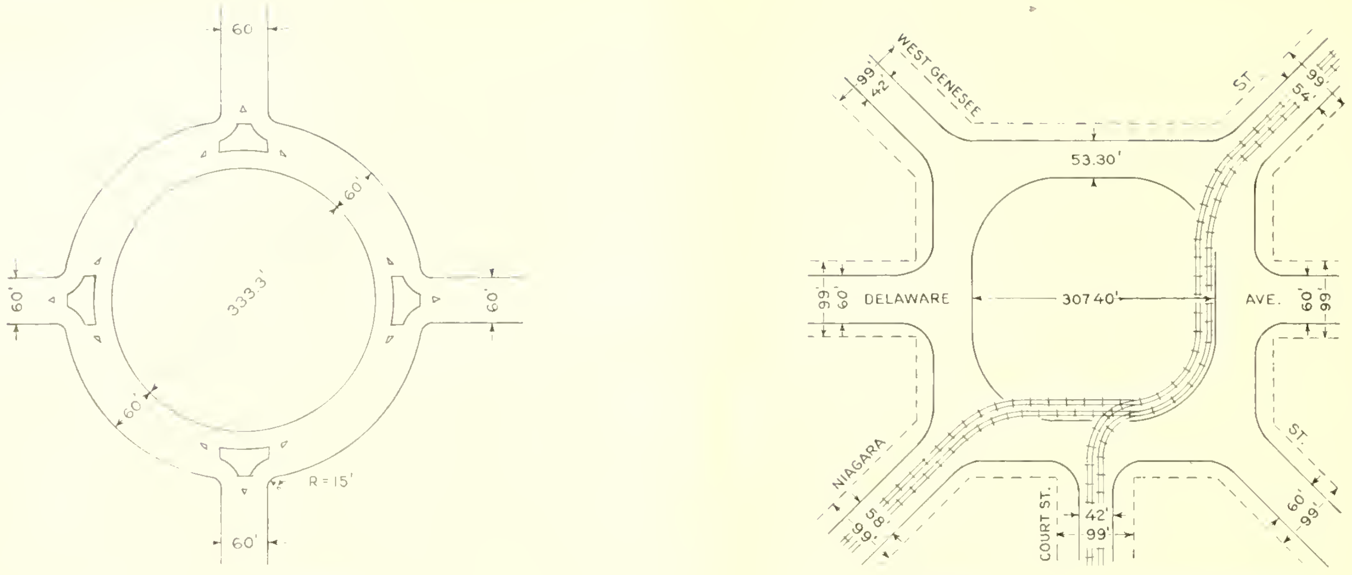
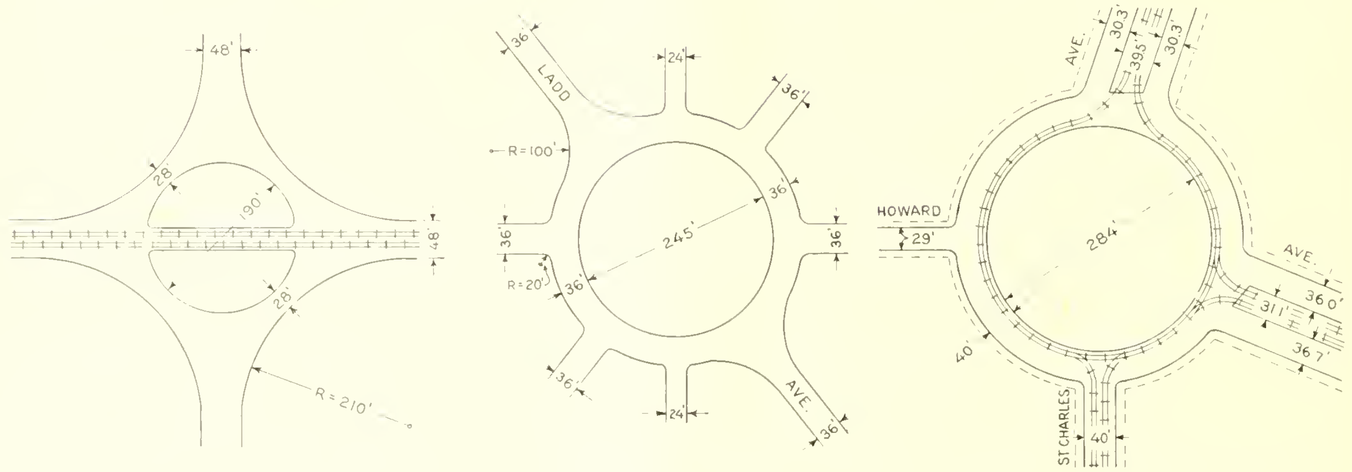
[1] Information furnished by Mr. H. B. Steeg, secretary-engineer, city planning commission.

[2] Report of the Committee on Street Traffic Economics, Bulletin 104 of the American Railway Association, p. 28.



A. MONUMENT PLACE, INDIANAPOLIS, INDIANA.

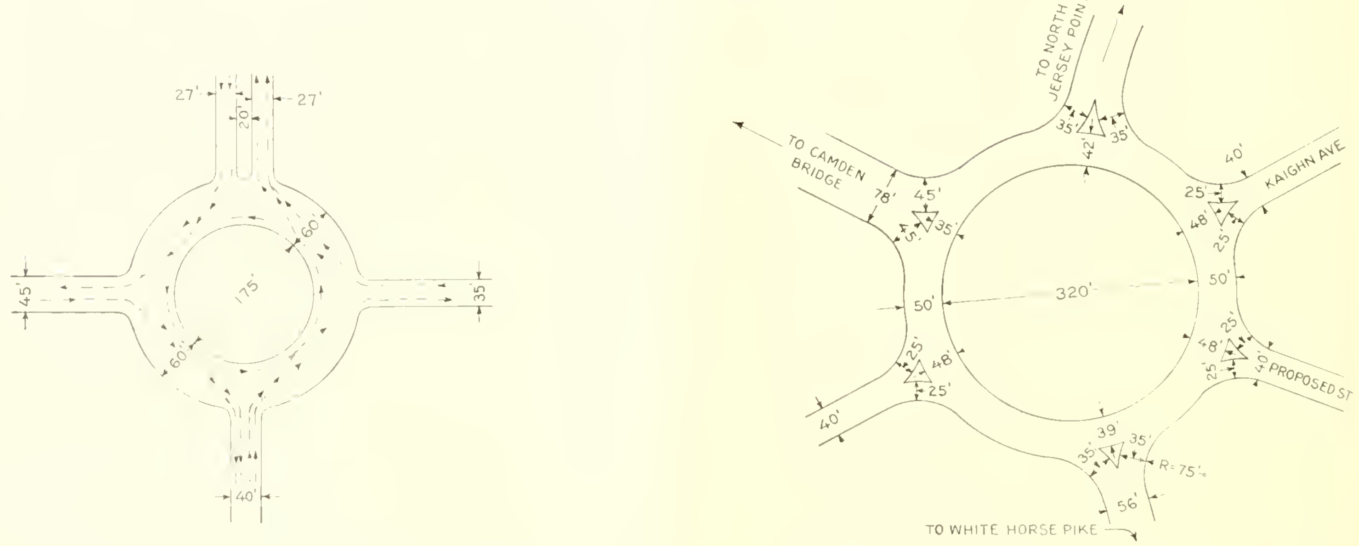
B. NIAGARA SQUARE INTERSECTION, BUFFALO, NEW YORK.



C. TRAFFIC CIRCLE IN PORTLAND, OREGON.

D. TRAFFIC CIRCLE IN CENTRAL PARK, PORTLAND, OREGON

E. LEE CIRCLE, NEW ORLEANS, LOUISIANA



F. COLUMBIA ROAD CIRCLE, BOSTON, MASSACHUSETTS.

G. KAIGHN AVE. ROND POINT, NEAR CAMDEN, NEW JERSEY.

FIGURE 8.—EXAMPLES OF EXISTING TRAFFIC CIRCLES

In both these designs excellent results are obtained with a roadway width less than the theoretical maximum, three lanes of traffic being provided in the first case and four in the second. Directional islands might well be added in the design, being particularly adapted in the first case where the open space in the entering streets created by the large curb radii is likely to prove confusing to entering traffic and hazardous to pedestrians.

Lee Circle.—Figure 8, E is a drawing of Lee Circle in New Orleans, La., which has a diameter of 280 feet. The short curb radii necessitate a reverse curve movement through the circle. This condition gives rise to a large area of unused pavement and makes improbable the desirable flat angle of intersection. However, the large diameter affords a greater interweaving distance, a desirable feature.

Traffic counts taken for the entire circle show a total of 2,543 vehicles entering the circle during one hour. Partial counts during the peak period indicated that nearly 3,400 vehicles passed through the circle during the rush hour. Speed studies showed that the speed varied from 16 to 19 miles per hour.

The roadway width in Lee Circle is 40 feet which, with a parking lane, permits three lanes of traffic. The total number of lanes on the entering streets, subtracting parking lanes, is 13. Thus the width is slightly under the maximum given by the formula suggested. The fact that there was at all times a complete freedom of movement in spite of the disadvantage of the short curb radii indicates that the formula gives a reasonable maximum width of roadway.

Columbia Road Circle.—This circle, near Boston, Mass., is represented in Figure 8, F. It is very near the minimum recommended design, having a central island 175 feet in diameter. It is located at the intersection of Old Colony Boulevard and Columbia Road and at certain times of day is required to accommodate a large number of vehicles with a large percentage of turning movements.

The effective operation of Columbia Road Circle, even under its peak loads, is a proof of the efficiency of this type of design. Observations indicate that well over 3,000 vehicles pass through the intersection during a peak hour at speeds ranging from 15 to over 20 miles per hour. During these observations congestion was not appreciable, no actual stoppage was ever seen, and little slowing down below the speed required for comfort in turning was noticed, even in periods of heavy traffic.

This circle further indicates the unused area created by the circular outside edge of the roadway. Traffic patterns on the pavement show that over 20 per cent of the pavement is never traveled over by vehicles passing through the circle.

Kaign Avenue Rond Point.—This circle, shown in Figure 8, G, was constructed by the New Jersey State Highway Commission. It has for a rotor a perfect circle, the large radius being necessary to provide for a proper interweaving distance.

It will be noticed that close attention has been paid to detail and that all details, in themselves correct, are properly combined to form an efficient intersection design. It is this attention to detail that premits the effective operation of this circle under the conditions of heavy traffic, a maximum of about 3,000 vehicles per hour being passed with little delay.

In the preceding designs no effort was made to provide for unbalanced traffic flow. The designs shown in Figure 9 are intended to illustrate the flexibility of the system, to suggest a modification applicable to the intersection where it is advisable to give preference to the traffic upon one or the other of the intersecting streets, and to suggest a rotary treatment for the common forked intersection.

It will be noted that each of the designs applies the basic principles on curb radii, island location, and interweaving distance previously developed. Figure 9, C shows how even a common parkway may be developed on rotary principle, which results in a design similar to that applied by Mr. Fritz Malcher in Radburn, N. J.[3]

SPECIAL DESIGNS WITHOUT ROTARY PRINCIPLE

The rotary principle is a specialized type of treatment which is particularly well adapted when a large volume of traffic must be handled at a comparatively low rate of speed. At intersections where there is only an intermittent flow of cross traffic, the vehicles upon the main highway are unduly penalized by the shape of the rotor. While the rotary system is not well adapted to such a situation, some form of treatment is desirable to permit the high speed boulevard traffic to flow without interruption and at the same time to protect the traffic crossing the main route and that turning into it from the minor route. The following designs have accordingly been presented to illustrate how this purpose may be accomplished by means of a system of well-placed channelizing islands.

The design in Figure 10, A, is well adapted to a T intersection where the traffic entering the heavily traveled street is very light. This type of intersection was developed by the Westchester County Park Commission and has proved very efficient at various intersections on the Saw Mill River Parkway. In this design the main roadway is 40 feet in width and, as shown, the opposing traffic is separated by islands at the intersection and the half roadway width increased to 30 feet. The widening effect is accomplished by means of reverse curves so flat (8° in this case) that there is no cause for reduction of speed on the main route.

The purpose of the islands is not to produce a rotary movement, but rather to provide a safety zone for vehicles waiting to enter or leave the main traffic by means of a left turn. For right turns, either entering or leaving the side street, 40-foot curves are provided and the increase from 20 to 30 feet in the half roadway width permits the turning vehicle to proceed alongside the main stream of traffic until it can accommodate itself to the speed of this traffic. In the case of left turns the importance of the islands is shown to best advantage. In turning left into the through street, the driver waits in the entering street until there is a sufficient interval in the traffic stream to allow him to cross to the space between the islands. If necessary, he may wait again in this protected area until the lane next to the island is open, when he enters and remains close to the island as he accelerates to the speed of the main traffic. During this acceleration period, there is still 20 feet to the right of the vehicle as it follows the island and there is ample room for all vehicles to pass even though both lanes are flowing full.

Similarly, in making a left turn from the main street into the side street, the driver remains close to the other island as he decelerates, thus leaving room for two lanes to pass on his right. Having slowed down suffi-

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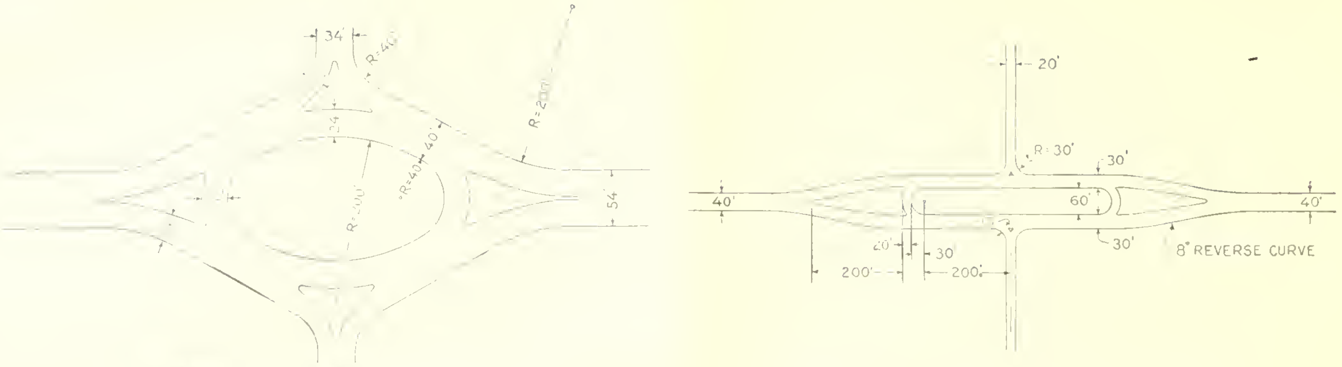
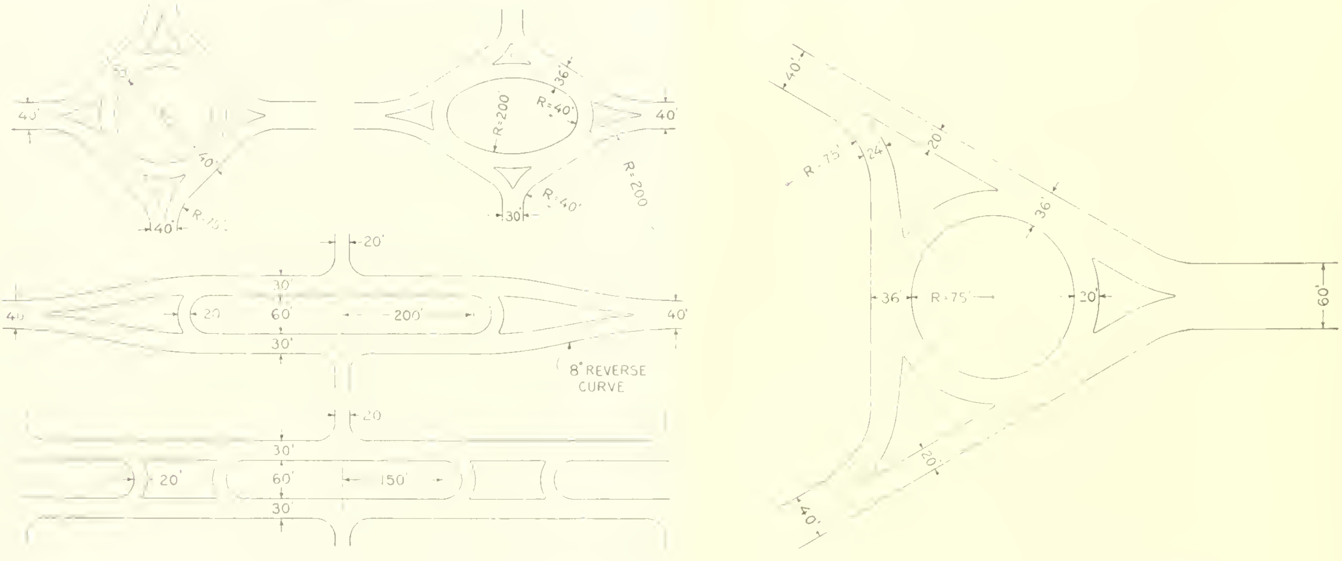
[3] Abolishing Street Traffic Intersections Without Grade Separation by Fritz Malcher, American City, September and October, 1929.



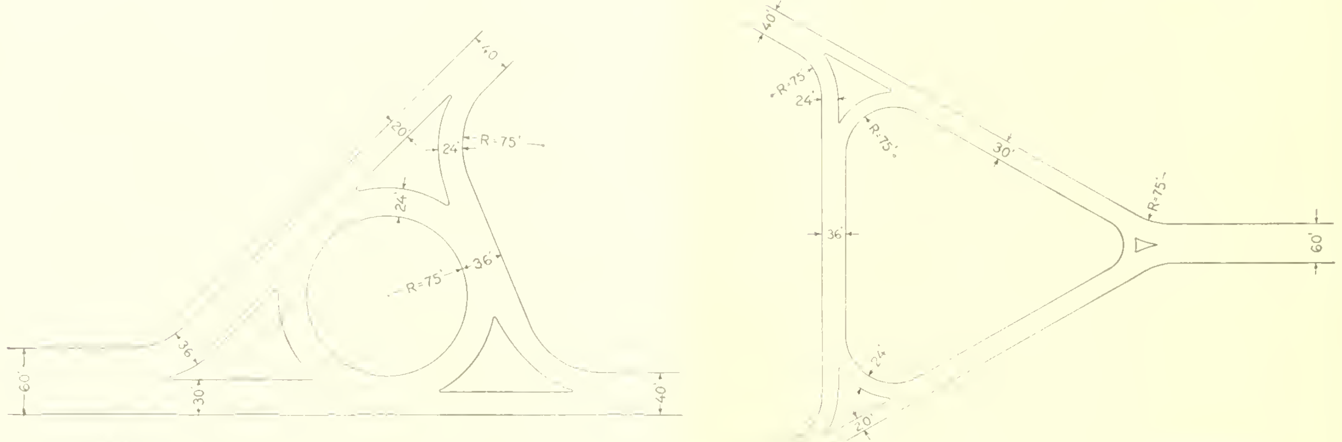
A DESIGN FOR ROADS OF UNEQUAL CAPACITY

B INTERSECTION OF A THROUGH ROAD WITH A MINOR ROAD



C DEVELOPMENT OF A PARKWAY FROM A ROTARY TRAFFIC CIRCLE

D ROTARY TRAFFIC DESIGN APPLIED TO A Y INTERSECTION



E FORKED INTERSECTION WHERE BOTH STREETS ARE OF EQUAL IMPORTANCE

F. SUBSTITUTION OF A CONNECTING ROADWAY FOR A ROTARY TRAFFIC CIRCLE

FIGURE 9.—SPECIAL ADAPTATIONS OF THE ROTARY PRINCIPLE

ciently, he turns into the protected area and as soon as the opportunity arises, crosses the other half of the highway and proceeds on the side street.

A design of this sort is less rigid in its various requirements than is a rotary traffic circle. Since no one will turn completely around an island, there is no necessity for making it the usual 60 feet in width. It is sufficient to place it so that it will be just inside the path followed by a vehicle in making a normal left turn as indicated by the traffic movements. Thus, the width is a factor only of the space required to provide an adequate safety zone. In this case 14 feet is used, a width which has been found satisfactory. Since a vehicle waiting in the protected area will stop at some point in its normal turning path, it will be in a diagonal position to the roadway at an angle of approximately 45°, so that 14 feet is ample to protect the largest car. The width of the narrow end of the island and its length are immaterial, the sides of the island generally being concentric with the sides of the road and the limiting condition at the narrow end being the width necessary to protect properly a significant warning sign.

No extra land is required over the normal right of way. An estimate of the quantities follows:

Item	Special design	Ordinary intersection	Excess
	Square yards	*Square yards*	*Square yards*
Grading	3,670	2,570	1,100
Paving	3,470	2,570	900
Island area	200	0	200
Curbing	[1] 435	0	[1] 435

[1] Linear feet.

The design shown in Figure 10, B, is very much the same as that discussed for stub-end intersections. This design, it may be observed, is applicable to the intersection resulting when a lightly-traveled street enters a heavily-traveled main highway on a curve and approximately tangent to the curve, a case very often encountered.

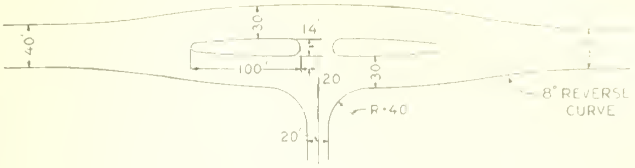
The physical features of this intersection follow exactly the same principles as those developed for the T intersection. In this case, the desired extra width is obtained by flattening the curve of the inside edge, and providing curves of greater degree on the outer edge. To eliminate a reverse curve on entering the main road from the secondary road and to prevent a vehicle from encroaching on the traffic lanes of the main road before it has acquired sufficient speed, the outer edge of the secondary road should be brought in on a tangent to the widened curve from a point some distance back of the intersection as shown in the diagram.

The effectiveness of the operation of this intersection depends entirely upon the proper location of the islands which, in turn, depends upon the radius of curvature and the angle of intersection. The details of the location of these islands may be seen in the diagram.

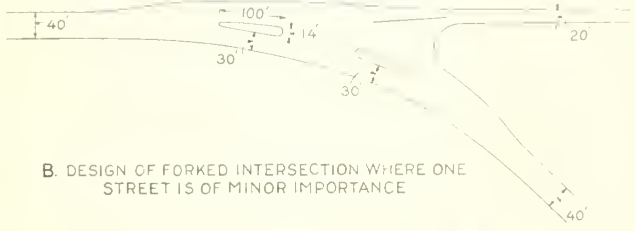
With the usual rights of way now acquired for rural highways, this design does not require much excess land. Assuming 100 and 60 foot rights of way, respectively, and that the present roadways are centered on the center lines of the rights of way, 2,500 square feet of excess land will be required, consisting of a small triangle in the throat of the intersection. When the design is to be constructed in conjunction with a relocation of the highways under consideration or in conjunction with new construction, the center lines of the highways may be so placed with respect to the rights of way that no extra land is required.

The design in Figure 10, C, another adaptation of the Westchester system, is similar to that presented in Figure 10, B except that the minor street is the one requiring the turn instead of the major street. The basic principle is the same; the turning traffic is protected from the through traffic and can not inconvenience it. Extra highway width on either side of the islands permits the slower turning traffic to mingle freely with the fast through traffic and an ample protected area between the islands is provided for waiting if necessary before turning.

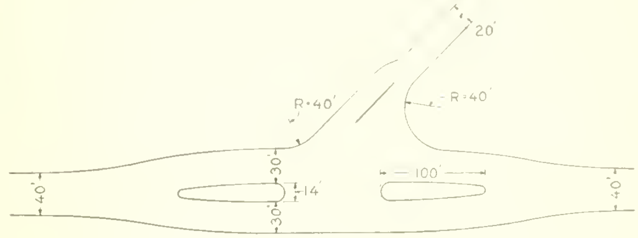
The feature to be noted is the difference in the location of the safety islands. As in the previous design, they are so located that traffic turning about them will be directed into the proper route, but because of the difference in angles and the straight instead of curving through highway, the distance between the islands is much less. This indicates the necessity of a study of the particular intersection before placing the islands, since their proper location is the determining factor in the successful operation of this type of intersection treatment.



A. INTERSECTION OF A THROUGH STREET WITH A MINOR STREET



B. DESIGN OF FORKED INTERSECTION WHERE ONE STREET IS OF MINOR IMPORTANCE



C. DESIGN OF FORKED INTERSECTION WHERE ONE STREET IS OF MINOR IMPORTANCE

FIGURE 10.—SPECIAL DESIGNS WITHOUT ROTARY PRINCIPLE

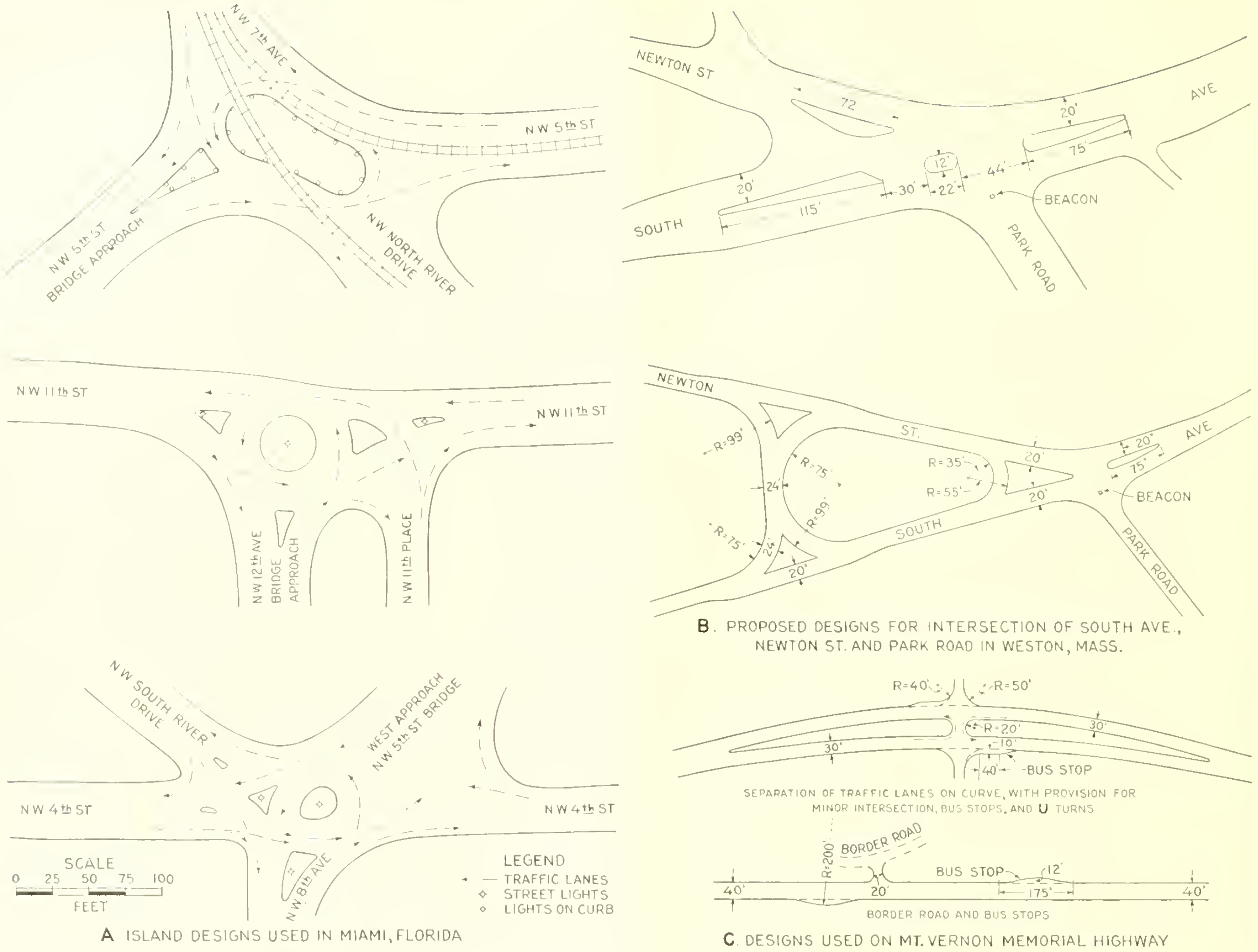
The extra land required is negligible and, as the additional pavement is dependent on the angle of intersection, no specific estimate is presented. In any case, the quantities will not be much in excess of those for the stub-end, right-angled intersection. (Fig. 10, A.)

EXISTING SPECIAL DESIGNS

The designs presented in the preceding paragraphs were not intended to cover all possibilities, but rather to bring out the principles involved and to show their application to a few common types of intersection. The designs illustrated in Figure 11 show how, in some cases, these principles have been actually applied and, in others, the proposed application. Each design represents a carefully considered solution of the particular problem and is illustrative of a practice which might well be applied at countless intersections.

The three designs shown in Figure 11 A are in Miami, Fla., and represent a good application of the principle that the driver should be directed into the proper route by making that route the natural one. The first of these (N.W. Fifth Street) might be termed a modified rotary system, in which case the central island would be a rotor, about which the streams of traffic flow, vehicles entering or leaving as the line progresses. In a sense this is what actually happens, but because of the general layout of the intersection and the turning movements to be expected, it is probable that most of the traffic will turn about the island rather than follow it closely for any considerable distance, as in a true rotary movement. For this reason, therefore, this design is classed as a special design and all the islands are termed directional islands.

The traffic entering the intersection from any street is confronted by the oval island, about which it must turn. The island is so placed that any vehicle following its natural course through the intersection inter-



A ISLAND DESIGNS USED IN MIAMI, FLORIDA

B. PROPOSED DESIGNS FOR INTERSECTION OF SOUTH AVE., NEWTON ST. AND PARK ROAD IN WESTON, MASS.

SEPARATION OF TRAFFIC LANES ON CURVE, WITH PROVISION FOR MINOR INTERSECTION, BUS STOPS, AND U TURNS

BORDER ROAD AND BUS STOPS

C DESIGNS USED ON MT. VERNON MEMORIAL HIGHWAY

Figure 11.—Existing Special Designs

sects other lines of traffic at an oblique angle, which makes interweaving or intermingling easy.

The second design (N.W. Eleventh Street) represents the solution of a rather complicated intersection, the open areas of which, without directional islands, would cause confusion. The largest of these islands is a circle 40 feet in diameter. As in the preceding case, this might be termed a rotary traffic island, but its small size and the fact that again few vehicles follow close to the edge for any considerable distance leads to its classification as a directional island.

Its location is the most important feature of this design, for upon this depends the routing of all traffic passing through the circle. The other islands serve to make vehicles approach the circular island at the proper angle. The two smaller islands at the east of the central island provide safety for traffic turning left from NW Eleventh Place, and prevent any vehicles from cutting to the left of the circle as they emerge from NW Eleventh Street.

The third design (NW. Fourth Street) is composed of strictly directional islands, there being no semblance of rotary movement. The islands are so shaped and located that the route which any vehicle should follow is obvious and is the one which will cause the minimum of confusion in the intersection area.

In all these cases the islands are of permanent construction, all being curbed and planted with grass, shrubs, or palms which add measurably to their appearance. It is of interest to note that they were not installed as permanent fixtures until the size and shape had been determined by trial, using temporary islands. In this way it was possible to insure efficient operation.

The following excerpt from a letter from Mr. J. E. Jewett, office engineer, is of interest:

> There is little question but what such traffic islands reduce accidents very materially. Rotary traffic is usually slow traffic in terms of miles per hour, but is rapid traffic in terms of vehicles per hour. As traffic congestions develop, we expect to install more such traffic separators.
>
> We have learned that we can seldom lay out a system of islands in the office that will work. It is nearly always necessary to make changes in the field. Our policy is to put up movable forms for the islands, making changes here and there until we are convinced that we have solved the problems. These forms are left in place as long as 30 days before the permanent curbs are built.

Figure 11, B represents two designs prepared for the intersection of South Avenue and Newton Street in Weston, Mass. The upper plan shows an adaptation of the design presented as the Westchester type to the special conditions encountered here. The islands are so shaped and located that they direct traffic into the routes consistent with the most efficient operation of the intersection. No traffic lines are materially deviated from their natural courses and ample area between islands provides for the safety of the turning traffic.

A more elaborate solution, which would be justified only in case of a marked increase of traffic on both routes, such that the consequent delay is considerable, is seen in the lower plan. This design, similar to Figure 9, E provides a rotary movement, and necessarily functions well only at relatively low speeds.

It should be noted that the second design might be considered an extension of the first, the throat of the intersection being practically the same in both cases. Allowance in a design for future development is an important feature.

PROGRESSIVE DEVELOPMENT OF AN INTERSECTION

In view of the tremendous cost of acquisition of land at a date when traffic demands require an intersection treatment, it is an increasing practice to obtain control of sufficient land abutting intersections to allow progressive improvement.

Various means of reserving this land for future use are exemplified by the methods used in city planning control of new developments. The simplest and most effective method is by outright purchase at the time of initial construction, based on the theory that it costs little more to obtain a wide strip of land than a narrow one, but that to take additional land after the original route is laid out is invariably expensive.

An alternative is to purchase immediately only the land necessary for the first development of the intersection; and, by means of zoning regulations, to insure against private developments in the right of way to be needed in the future. Such regulations must necessarily conform to court decisions on the subject of zoning in the jurisdictions concerned. A reasonable restriction would allow the owner to make whatever use of his property he desired, so long as he complied with the general zoning requirements, but would stipulate that any buildings or other improvements would be removed from the area in question, without compensation to the owner, at the time anticipated for the improvement of the intersection. This would permit the erection of inexpensive buildings having a length of life less than the period between the first and the second improvements, as well as the use of the land for agricultural or other purposes. It would, on the other hand, prevent the building of permanent structures which would preclude the possibility of a grade separation or a traffic circle.

Where zoning acts do not permit this procedure, it may be possible to incorporate in the deed covering the original purchase a provision similar to the restriction outlined above, applying to land which will be required for future intersection development.

In case all the land required for full development of the intersection is purchased at the time of initial construction, the land not needed immediately may be leased for the construction of temporary buildings or such other use as may be profitable. In this way all or part of the carrying charges on the additional land may be defrayed during the period when it is not needed for the intersection.

If this plan were followed a State highway department or other agency would find itself with a considerable amount of land on its hands for which it had no particular use at the time but which must be kept in a presentable condition. In order best to use the available area, the following methods for a progressive development of an intersection are presented.

The intersections fall naturally into two classes according to location, first, those in country or parkway locations and, second, those in districts which will be built up with industrial or commercial structures.

In the first instance, a proper design would be one requiring a generally square plot of land with the intersecting roads forming its diagonals. To provide for any desired future construction, this square should be about 425 feet on a side, its corners being located on the center lines of the right of way about 300 feet from the intersection of the center lines. Assuming that this amount of land is taken at the intersection of two newly designed routes, it is of interest to follow the possibilities of the layout of the intersection through increasing traffic stages, as indicated in Figure 12.

The first design shows the crossing of two 20-foot roadways. The intersecting edges are rounded at a 30-foot radius, but otherwise nothing need be done. As the traffic becomes heavier or more flexibility of movement is desired, it may be desirable to increase the roadway widths to 40 feet or two lanes in each direction, but it may still be unnecessary to provide any special intersection treatment. If turning traffic becomes a factor and left turns as well as right turns complicate the traffic, turning roads may be beneficial. Roads such as these are now installed as a component

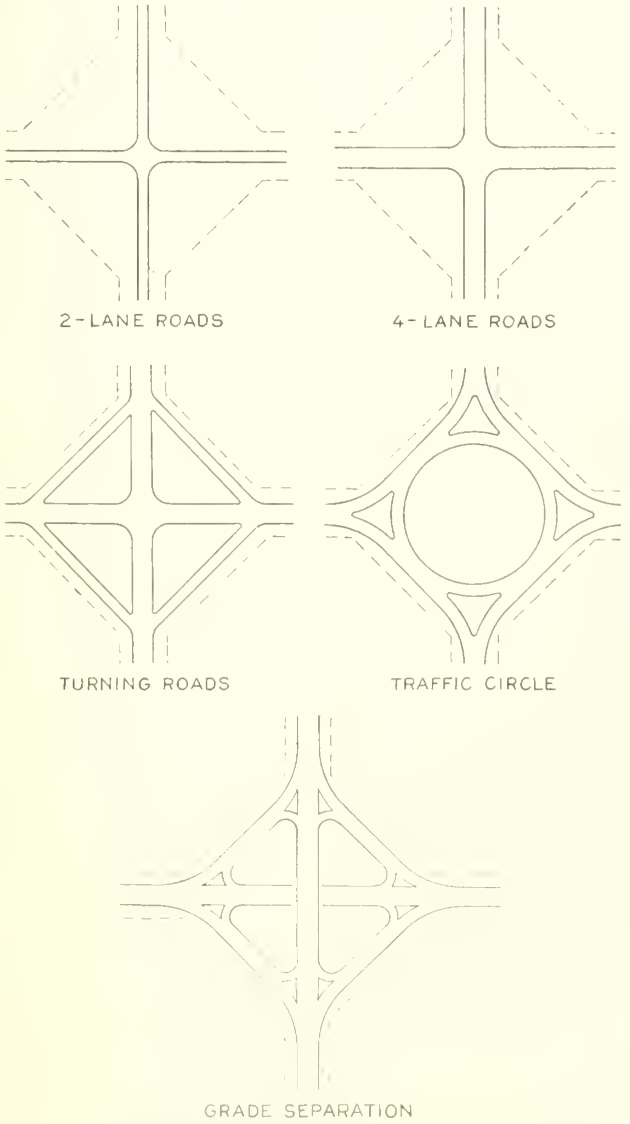


FIGURE 12. SUGGESTED DESIGN FOR THE PROGRESSIVE DEVELOPMENT OF AN INTERSECTION

part of many intersections, but usually are laid out in a circular arc, of perhaps 200-foot radius. The reason for selection of the straight turning roads is apparent, however, on inspection of the solution for the next stage, a rotary traffic circle.

When traffic becomes so heavy that traffic signals would be necessary to regulate the right-angled crossing, a rotary design is very effective and, in this case, inexpensive. The turning roads already constructed will form a part of the circular roadway and a central island of 150-feet radius may be laid out in the area inclosed by the turning roads. The radius of the throats of the entering streets will be increased, for though in the previous case only turning traffic used the diagonal roads, now all traffic swings right on entering the intersection and follows around the rotor. It may be seen that there is ample area to provide a traffic circle which will fulfill every specification developed in the section on rotary traffic.

In the case when the traffic demands become so great that the rotary traffic circle will cause an economic loss due to the required reduction of speed of vehicles passing through it, a grade separation may be justified. When this installation is completed, it will be found that the diagonal roadways which were first classified as turning roads and then as an integral part of the circular roadway of the traffic circle have now become access roads for a grade separation and the two intersecting streets again pass straight through the intersection, but at different elevations.

Figure 13 shows a clover-leaf grade separation at the intersection of U. S. Route 1 and the Mount Vernon Memorial Highway.

The foregoing discussion outlines a possible intersection development from the simplest to the most elaborate, each stage of which fits as nearly as possible with the next and eliminates almost altogether the too prevalent factor in highway work, depreciation through obsolescence. Proper foresight will eliminate much unnecessary expense in solutions which are worthless after a short period. It is by no means compulsory that each step be followed in order, but it is extremely unlikely that the traffic demands at any intersection will warrant a transition from the initial to the final solution in one step and if a progressive, flexible development in provided for, much expense and inconvenience may be saved.

The second case, that in which the intersection is in a district expected to be closely built up, does not so readily adapt itself to a progressive development. It is for an intersection such as this that the Cook County Grade Separation Committee recommended a set-back line 60 feet on either side of the center line for a distance of 660 feet back of the intersection. As in the preceding case it is not necessary to proceed at once from the simple intersection to the final solution, but rather to follow a series of steps which, though not developing one from the other as did those in the outlying intersection, have a definite relationship with one another.

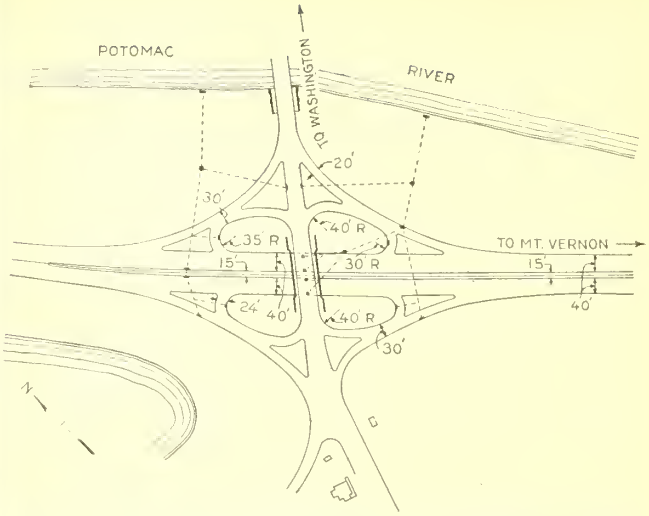


Figure 13.—Grade Separation at the Intersection of U. S. Route 1 with the Mount Vernon Memorial Highway

Again, there are the two narrow intersecting roadways and an equal or greater amount of excess land not in the form of a square, but instead shaped like a huge cross, the center lines of which are the center lines of the intersecting roadways. In this case, also, the second step may well be the widening of the two roadways to four lanes each, merely rounding the intersecting edges with 30-foot curves. As turning traffic needs to be better accommodated or as the necessary stoppage by control devices interferes with the freedom of right turns, a splaying design becomes appropriate. Here is an admirable opportunity for installing the separated lane splaying feature, since land is available both on the sides and for a sufficient distance back of the intersection. As the traffic becomes even heavier, a second line may be filled in between the through pavement and the turning lane to increase the storage capacity and in this way increase the efficiency of the intersection somewhat. Following this comes the final step, a complete separation, effected by carrying the middle four lanes of one roadway under the other.

In this development one feature does not become an integral part of the succeeding feature. The only possibility is that the turning lanes introduced by the splaying may serve as turning roadways for the separation. It is quite possible, however, during the construction of a project such as the grade separation structure and approach ramps that the existing pavement would be seriously injured and in this event the outer paving would also be relaid.

Thus, even though successive designs may not lead to the construction economies resulting from the parkway development, a proper program of construction related to the traffic needs will result in an intersection effective at all times.

Both the preceding cases are concerned with the intersection to two approximately equally traveled highways. Another case which very often arises is that of a major, heavily traveled, highway crossing several other highways carrying different volumes of traffic. This case, too, is well worth considering from the point of view of progressive developments. One or more of the intersecting highways may have traffic sufficient to warrant a grade separation or rotary traffic circle immediately while others, though not now critical, may in time carry such a volume of traffic that their intersections necessitate some special treatment and still others obviously may never have enough travel to warrant an elaborate intersection design. It will be found that in every case where the volume of traffic to be expected some years hence can be estimated, some progressive plan can be well adapted to the intersection. Various designs have been presented for the intersection of roadways having different volumes of traffic and all of these designs have been built up from the same fundamental principles; so it is only a method of applying the pertinent designs and providing a means of evolving one from the other as traffic increases.

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A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads was printed in Public Roads, vol. 13, No. 3, May, 1932. Copies of this list may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

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United States Department of Agriculture
Bureau of Public Roads

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF JUNE 30, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal Aid allotted	Percentage completed	Mileage			Estimated total cost	Federal Aid allotted	Mileage				
					Initial	Stage *	Total			Initial	Stage *	Total		
Alabama..........	2.393.9	$ 1.812.011.04	$ 884.109.47	95	74.1		74.1	$	$				$ 5.469.694.24	Alabama..........
Arizona..........	1.220.1	2.782.928.26	2.014.700.15	64	104.9	61.7	166.6	4.594.57	3.459.24		0.3	0.3	742.533.48	Arizona..........
Arkansas..........	1.960.2	2.452.768.24	1.127.328.92	82	64.7	45.1	109.8	404.816.95	201.796.90	17.4		17.4	2.162.507.23	Arkansas..........
California........	2,328.6	10.475.483.57	4,985.605.01	54	211.3	52.0	263.3	1.475.029.90	645.642.39	44.4	7.0	51.4	32.946.23	California........
Colorado..........	1.603.2	4.355.182.99	2.276.199.42	75	171.4	50.8	222.2	73.822.81	41,458.88	7.5		7.5	2,057.603.69	Colorado..........
Connecticut.......	281.7	4.362.762.05	1.613.060.26	74	30.2		30.2	563.726.40	263.977.96	7.7		7.7	390.452.51	Connecticut.......
Delaware..........	345.7	870.529.25	435.264.62	59	41.2		41.2	86.524.00	43.262.00	4.7		4.7	157.140.82	Delaware..........
Florida...........	638.0	3.563.837.58	1.659.559.53	85	102.1		102.1	655.532.89	327.766.42	17.3		17.3	2.541.174.33	Florida...........
Georgia...........	3.068.4	6.410.060.34	2.999.196.48	61	160.3	166.8	327.1	174.461.46	87.230.72	6.5	1.2	7.7	1.345.949.03	Georgia...........
Idaho.............	1.460.3	2.573.893.23	1.517.125.45	77	123.5	93.7	217.2	1.302.449.66	781.761.33	30.8	228.7	259.5	345.018.54	Idaho.............
Illinois..........	2.649.2	25.384.659.55	11.764.654.34	76	751.9	36.8	788.7	3.495,560.26	1,569,434.05	125.4	5.2	130.6	1.310.641.51	Illinois..........
Indiana...........	1.822.0	10.326.406.44	5.114.814.33	63	352.4	20.5	372.9	1.609.394.59	792.712.22	73.7		73.7	381.104.02	Indiana...........
Iowa..............	3.350.2	3.512,604.68	1.639.295.18	30	178.3	40.4	218.7	2.294,536.74	1.072.924.03	106.9	25.4	132.3	87.585.99	Iowa..............
Kansas............	3.643.0	5.160.560.54	2.521.332.53	64	293.4	68.2	361.6	2.324.281.14	1.126.345.63	72.3	304.6	376.9	567.001.31	Kansas............
Kentucky..........	1.911.0	2.439.460.44	1.079,480.86	74	144.2	4.0	148.2	1.399,504.67	525.166.87	17.3	107.8	125.1	1.182.786.01	Kentucky..........
Louisiana.........	1.566.8	7.471.657.46	3.511.759.18	46	87.0	10.6	97.6	1.103.383.51	538.516.32	29.8	13.3	43.1	321,408.48	Louisiana.........
Maine.............	720.4	3,348.521.01	1.423.360.93	52	82.9		82.9	887,397,91	431.757.49	34.3		34.3	137.272.92	Maine.............
Maryland..........	778.9	701.917.80	255,754.9	81	23.4	.6	24.0	1.633.360.88	691.459.64	60.4	5.6	66.0	8.964.30	Maryland..........
Massachusetts.....	817.3	7.129,265.67	2.897.641.92	68	71.3	.2	71.5						602.611.77	Massachusetts.....
Michigan..........	2.071.5	9,062,647.19	4.162.775.75	70	395.4	31.9	427.3	655.889.46	325.630.00	21.4	20.1	41.5	2.452.326.98	Michigan..........
Minnesota.........	3.956.1	11.284.587.17	3.749.501.38	33	204.9	296.8	501.7	842.326.55	229.800.00	15.7	20.2	35.9		Minnesota.........
Mississippi.......	1.809.0	3.910.633.74	1,915.689.81	85	171.2	68.8	240.0	116.017.23	58.008.61	16.1		16.1	5.152.976.98	Mississippi.......
Missouri..........	2.922.0	7,031,555.37	3.036.644.01	41	191.0	88.3	279.3	689.291.30	329.813.03	25.1	11.9	37.0	567.602.87	Missouri..........
Montana...........	2.724.3	4.576,181.82	2.572.131.43	75	397.7	31.5	429.2	67.501.64	36.033.68	5.7	4.2	9.9	3.465.267.40	Montana...........
Nebraska..........	4,255.2	3,329.003.58	1,650.271.38	73	151.9	26.1	178.0	1.817.311.97	845.048.85	28.1	111.6	139.7	1,849.946.51	Nebraska..........
Nevada............	1,281.2	1.576.562.30	1,289.906.62	81	21.2	163.0	184.2	859.152.00	693.501.34	26.8	33.3	60.1	86,974.21	Nevada............
New Hampshire.....	421.0	339.864.67	140,869.77	85	9.5	2.5	12.0						553.268.42	New Hampshire.....
New Jersey........	616.5	4,716,807.19	1.770.638.23	56	34.3	.5	34.8	1.023.047.07	496,650.88	.9		.9	698,945.91	New Jersey........
New Mexico........	2.250.1	1.469.046.56	1.028.823.23	92	61.9	17.8	79.7	998.601.99	618.039.80	41.1	42.3	83.4	897.851.56	New Mexico........
New York..........	3.273.4	14.125.400.00	6.352.125.00	50	334.3		334.3	6.402.600.00	2.462.500.00	135.9		135.9	2.250,042.54	New York..........
North Carolina....	2.226.4	1.152.326.37	567.278.11	77	67.6	5.0	72.6	300.312.76	150.156.36	7.1	4.0	11.1	3.728.867.35	North Carolina....
North Dakota......	5.062.7	3,316.919.40	1.680.534.96	45	383.7	283.5	667.2	1,931.160.99	973.516.22	183.5	478.4	661.9	821.723.31	North Dakota......
Ohio..............	2.858.4	6.314.566.20	2,097.263.73	79	101.7	26.4	128.1	6.292.605.00	2.240.240.53	153.0	46.4	199.4	1.682.314.29	Ohio..............
Oklahoma..........	2.295.2	3,478.222.78	1.625.741.63	84	157.3	45.0	202.3	1.713.059.54	877.359.08	112.5	17.6	130.1	1.276.063.13	Oklahoma..........
Oregon............	1.522.8	4.145.699.29	2.305.513.64	85	128.0	47.1	175.1	1.590.510.22	864.268.36	46.9	74.8	121.7	875.392.92	Oregon............
Pennsylvania......	3,012.6	5.021.298.77	2.322.443.57	35	181.7		181.7	6,392.381.18	2,851.912.07	196.7		196.7	775.956.02	Pennsylvania......
Rhode Island......	255.6	926.504.32	459.140.00	60	16.7	2.2	18.9						325.565.85	Rhode Island......
South Carolina....	2.013.6	3.131.546.27	1.409.414.06	94	73.2	59.7	132.9	101.152.78	50.000.00	5.2		5.2	744.636.09	South Carolina....
South Dakota......	4.039.4	3.318.617.13	1,909.863.38	70	266.1	183.8	449.9	233.615.05	157.987.24	30.8	12.0	42.8	1.085.351.79	South Dakota......
Tennessee.........	1.682.4	1.427.533.89	697.948.21	57	48.7	6.4	55.1	733.446.20	356,461.69	17.1	21.8	38.9	3.157.110.04	Tennessee.........
Texas.............	7.802.7	13.843.539.36	6.348,479.78	69	605.0	223.6	828.6	3.980.124.61	1,802.196.04	232.1	185.1	417.2	3.333,824.82	Texas.............
Utah..............	1.203.5	1.109.608.87	797.552.81	67	84.9	27.7	112.6	847.650.66	635.771.[illegible]2	37.6	108.5	146.1	520.628.78	Utah..............
Vermont...........	339.1	1.408.800.97	630.501.86	19	45.6		45.6	7.618.95	3.809.47					Vermont...........
Virginia..........	1.922.9	1.915.279.75	957,639.81	69	113.5	6.2	119.7	754.053.46	298.883.54	22.9	21.5	44.4	1.817.277.70	Virginia..........
Washington........	1.197.9	2.915.547.20	1.432.094.35	62	106.2	14.5	120.7	519.265.07	213.800.00	11.5	5.3	16.8	1.007.069.51	Washington........
West Virginia.....	893.3	1,783.964.32	768.721.07	71	44.5	10.5	55.0	283.803.60	132.744.79	6.2		6.2	998.160.51	West Virginia.....
Wisconsin.........	2.593.8	7.833.923.20	3.203.398.32	41	204.8	75.4	280.2	1.110.874.93	432.400.00	29.6	23.5	53.1	283.938.52	Wisconsin.........
Wyoming...........	1.894.3	3.630.824.26	2.232.041.40	72	192.4	230.9	423.3	205.019.30	134,595.13	26.2	20.1	46.3	120.711.31	Wyoming...........
Hawaii............	76.3	811.202.36	395.047.85	65	22.0		22.0	740.008.72	598.548.44	18.8	3.6	22.4	1.335.908.87	Hawaii............
TOTALS............	101.032.1	234.042,724.44	109.230.238.67	64	7.885.4	2.626.5	10.511.9	60.696.780.57	28.014.349.16	2.110.9	1.965.3	4,076.2	61.720.100.60	TOTALS...........
Construction completed...		148.147.000	69,982,000											
Balance Uncompleted......		85.896.000	39.248.000											

(*) The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE. 1932

A22.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 6 AUGUST, 1932

A BITUMINOUS MACADAM ROAD AFTER 21 YEARS OF SERVICE

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 6 August, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

RATIONALIZATION AND SIMPLIFICATION OF TEST REQUIREMENTS FOR LIQUID ASPHALTIC MATERIALS

By E. F. KELLEY, Chief Division of Tests, U. S. Bureau of Public Roads, and PREVOST HUBBARD, Chemical Engineer, The Asphalt Institute.

THE PURPOSE of test requirements in specifications is to insure that materials furnished to meet the specifications will be suitable for the use for which they are intended. The value of test requirements is measured by the extent to which they fulfill this purpose.

Test requirements may be of two types—first, those which describe the properties or characteristics of a product which it is essential that it possess if it is to be suitable for a given purpose and, second, those of a restrictive nature which define characteristics which may or may not be inherent in a suitable product, and the presence or absence of which has no measurable influence on satisfactory performance.

It is evident that, so far as possible, tests of the first type only should be used in specifications. However, the selection of such tests is not always a simple matter because of the lack of sufficient data, and the situation is frequently complicated by the difficulty of eliminating tests of the second class. The latter serve no useful purpose and such prestige as they may enjoy is undeserved, but before they can be discarded it is commonly necessary to overcome the inertia due to years of unquestioned acceptance.

Unfortunately, many of the common test requirements for road materials are of the second class. Often of obscure origin and possibly developed originally for a purpose foreign to their present use, they retain their place in specifications, sometimes for lack of adequate substitutes and sometimes because of established precedent. This is particularly true of the specifications for that class of road materials known as liquid asphaltic products, of which enormous quantities are now used in the construction and maintenance of low-cost roads. Many of the tests which are used at present in specifications for these materials have been handed down from specifications for bituminous materials of an entirely different class, often semi-solid in character, which were never intended to be used for the same purposes as those to which liquid asphaltic products commonly are put in highway work to-day.

In any attempt to rationalize specifications each individual test must be carefully scrutinized to ascertain its actual value. What does the property defined by the test signify in connection with the use to which the material is to be put and how can this property be adequately controlled by use of the test? If satisfactory answers to these two questions can not be formulated, the test has no place in a specification. Many of the tests now in common use will fail when subjected to this scrutiny.

In addition to the need for selecting the tests for liquid bituminous materials on a rational basis, there has been an equally urgent need to simplify the test requirements in current use.

The lack of agreement among the various State highway departments with respect to tests and methods of testing has created a condition which has been little short of chaotic. It is unnecessary to go into details here as this has already been done.[1] It is sufficient to state that a survey in 1930 of State highway department specifications for liquid asphaltic products showed that there were in use, including all variations, a total of 119 different tests. As the practice of the different States in specifying such products varied greatly with respect to the selection of tests required, it was too often impossible to compare the specifications of two States in order to ascertain whether they were using practically the same or entirely different materials for the same type of work.

In addition to the complications entailed by the practice of specifying different tests or combinations of tests to define the characteristics of materials of essentially similar character, there also has been no general agreement regarding the test results, or test limits, which should be incorporated in specifications. The net effect of the non-uniform practice with respect to the requirements for both tests and test results has been the establishment of an unduly large number of grades of material. This has placed on manufacturers the burden either of carrying a large stock of different grades or of being prepared to manufacture, on short notice, materials to meet the wide variations required by the specifications of the different states. Naturally, this has resulted in increased expense which ultimately has been charged to the consumer.

The disadvantages of the existing situation were manifest to all concerned and as a result there was inaugurated a cooperative undertaking designed both to simplify and to rationalize specifications for liquid asphaltic materials. This cooperation involved the highway departments of the various States, the asphalt industry as represented by the Asphalt Institute, and the Bureau of Public Roads.

STATES GROUPED BY REGIONS FOR PURPOSES OF STUDY

For convenience in carrying on the cooperative program the States were divided into five groups or regions, the division being made largely on the basis of the territory served by the principal groups of producers. The States in each region are as follows:

Region No. 1.—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut, New York.

Region No. 2.—New Jersey, Pennsylvania, Delaware, Maryland, Ohio, West Virginia, Virginia, North Carolina, South Carolina, Georgia, Florida.

Region No. 3.—Michigan, Indiana, Illinois, Wisconsin, Minnesota, Iowa, Missouri, Nebraska, Kansas, Oklahoma.

Region No. 4.—Kentucky, Tennessee, Alabama, Mississippi, Arkansas, Louisiana, Texas.

Region No. 5.—North Dakota, South Dakota, Montana, Wyoming, Colorado, New Mexico, Idaho, Utah, Arizona, Nevada, Washington, Oregon, California.

As a preliminary to the cooperative simplification work it was first necessary to establish a basis for the comparison of the great variety of requirements in current use. Without such a basis of comparison there is no adequate means for establishing the value of present and proposed tests nor for comparing the specifications of one State with those of another, even for materials which may be essentially the same.

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[1] Cooperative Research and Standardization of Low Cost Road Improvement, by Prevost Hubbard, Eighth Annual Asphalt Paving Conference, 1929. Simplified Specifications for Liquid Asphaltic Materials, by E. F. Kelley, Ninth Annual Asphalt Paving Conference, 1930. Cooperative Simplification of Tests and Specifications of Liquid Asphaltic Materials, by J. T. Pauls, Association of Asphalt Paving Technologists, January, 1932.

SIMPLIFIED SCHEME OF ANALYSIS ADOPTED

To furnish this basis of comparison a definite test procedure for the examination of all liquid asphaltic products was proposed. This procedure, known as the simplified scheme of analysis, was designed to reduce the required number of tests for any product to a minimum and the attempt was made to select the tests on a rational basis so that, by proper selection, they would define the essential characteristics of all materials of this general class.

Meetings, which were attended by representatives of the cooperating agencies, were held in the five regions early in 1931 for the purpose of considering the proposed simplified scheme of analysis and to formulate a program for the future conduct of the cooperative work.

As a result of these meetings the scheme of analysis was modified in some of its details and adopted for use in the investigation, for which a program was drawn up. This investigation was for the two-fold purpose of studying the adequacy of the scheme of analysis and developing data which would permit a comparison of different specifications and the materials furnished to meet them. The program provided:

(*a*) That the producers should submit to the State highway departments samples of their products which would meet State specifications in which they were interested commercially. Two samples, varying as much in consistency as possible while still complying with all specification requirements, were to be submitted.

(*b*) That each sample should be tested, by the producer and by the department to which it was submitted, in accordance with the requirements of the State specifications and also in accordance with the approved scheme of analysis.

(*c*) That each State highway department should test, both according to the approved scheme and for compliance with its own specifications, a sufficient number of deliveries of materials purchased for use to cover the range in characteristics likely to occur in each type of material.

The cooperative work contemplated in this program was begun immediately after the regional meetings and was actively prosecuted until its completion late in the year 1931. In connection with this work the producers submitted to the State highway departments 1,154 samples of liquid asphaltic road materials. These samples were furnished by 27 producing companies from 47 refineries. In some cases a particular product complied with the requirements of several States so that with respect to the number of different materials there were some duplications. Eliminating such duplications, the samples submitted represented 715 different products produced to meet approximately 200 State specifications for all grades of liquid asphaltic materials used throughout the United States. The samples were all tested by the producers prior to shipment and a large percentage of them were tested by the highway departments to which they were sent. Thus, in addition to the basic data which were desired, there were made available a great number of check tests on duplicate samples of the same material and, in the cases of the duplication of materials mentioned above, check tests on the same materials by several laboratories. In addition to the tests made on samples furnished by producers, the State highway departments reported tests on 361 shipments of materials purchased for use.

The cooperative program provided that the producers should furnish to the Bureau of Public Roads, for such tests as might be considered necessary, duplicates of all samples of materials sent to the State highway departments. It also provided that copies of all test results should be sent to the bureau by the producers and by the States.

These duplicate samples were received by the bureau but were not tested. The samples will be retained for a reasonable period for possible use in connection with questions which may arise later.

The test reports were received by the bureau from time to time during the summer and fall of 1931. Late in the year, as the testing work neared completion, all of the data which they contained were placed in tabular form and distributed for the information of the cooperating agencies. The data contained in these tabulations give additional emphasis to the need for rationalization and simplification of the specification test requirements.

CLASSIFICATION OF MATERIALS BASED ON VISCOSITY AND DISTILLATION TESTS

The simplified scheme of analysis which has been followed in the cooperative work embodies two tests which are considered to be of major importance. These are the viscosity test and the distillation test. The viscosity test is made with the Saybolt-Furol viscosimeter at one of three selected temperatures, depending upon the character of the material and its intended use. The distillation test is included in the scheme of analysis to take the place of tests for determination of asphalt content and loss by volatilization at different temperatures. The residue from the distillation test is subjected to tests for consistency and solubility in carbon disulphide, replacing similar tests frequently made on original materials or on residues from the volatilization and asphalt content tests.

In tabulating the data obtained in the cooperative tests, the materials were classified as to type on the basis of these two important tests.

The materials were first divided into three classes, depending upon the consistency of the residue from the distillation test, as follows:

1. Liquid residue (float test less than 25 seconds at 122° F.)
2. Float residue (float test more than 25 seconds at 122° F. and penetration more than 300 at 77° F.).
3. Penetration residue (penetration less than 300 at 77° F.).

Each of these classes was further subdivided on the basis of viscosity, as follows:

A. Viscosity, Furol, determined at 77° F.
B. Viscosity, Furol, determined at 122° F.
C. Viscosity, Furol, determined at 210° F.

The nine types of material which result from this classification are shown in the following table:

Character of residue	Viscosity, Furol, at—		
	77° F.	122° F.	210° F.
1. Liquid	1-A	1-B	1-C
2. Float	2-A	2-B	2-C
3. Penetration	3-A	3-B	3-C

It is evident that materials of types 1-A, 1-B, and 1-C are of the nonhardening or extremely slowly hardening variety and, irrespective of their original viscosities, could not be expected to develop an asphalt cement in place in a road surface. Materials of types 2-A, 2-B, and 2-C yield a distillation residue too soft for a penetration test, but sufficiently viscous for a float test and therefore would be classed as slowly hardening products whose consistency might increase materially after application, but which could not be expected to develop an asphalt cement in place. Materials of types 3-A, 3-B, and 3-C, yielding a distillation residue sufficiently hard for a penetration test, would be classed as products which will eventually develop an asphalt cement in place, the rapidity of hardening being indicated by the results of the distillation test.

A study of the tabulated data, which included also State specification requirements for the various materials, showed clearly that many specifications in common use do not make the important distinctions of the above classification. In a number of cases several materials of entirely different type were found to meet all the requirements of a given specification. For instance, it was found that certain materials of types 1-A, 2-A, and 3-A were submitted to meet the requirements of a specification for a certain material designated as 45 per cent asphaltic oil. The essential requirement of this specification was that the material should contain between 45 and 55 per cent of residue of 100 penetration. The three distinct types met this specification except that the material of type 3-A had an asphalt content slightly above the maximum specified. The specification failed so far in defining important characteristics, that materials which developed liquid residues, float residues, and possibly penetration residues in the distillation test could all comply with its requirements.

The tabulated data with respect to specifications showed clearly the necessity for discarding many tests in current use if any simplification is to be effected. As an example, in one region the 51 specifications for liquid asphaltic products required a total of 44 different tests.

Early in 1932 a second series of regional meetings was held to consider the data which had been developed by the cooperative tests and to obtain an expression from the States on the adoption of the proposed simplified scheme of analysis for use in future specifications and the elimination of other tests which might be considered unnecessary.

The proposed scheme of analysis has been modified slightly with respect to certain details in the method of making the distillation test, and the test for solubility in naphtha, which had been added to the original scheme in the first series of regional meetings, was eliminated by the majority of the States represented at the second series of meetings. Aside from these changes the simplified scheme of analysis, as considered at the second series of regional meetings, is substantially the same as the scheme in its present recommended form, which is given in the following paragraphs.

RECOMMENDED SIMPLIFIED SCHEME OF ANALYSIS FOR USE IN SPECIFICATIONS FOR LIQUID ASPHALTIC ROAD MATERIALS

1. Flash point:
    - *a.* Cleveland open cup for materials having a flash point of 175° F. or more. A. S. T. M. standard method D 92-24.
    - *b.* Tagliabue open cup [2] for materials having a flash point of less than 175° F. Method approved by the Bureau of Explosives, 30 Vesey Street, New York City, and by the Interstate Commerce Commission (Regulations for Transportation of Explosives, etc., par. 227).
2. Consistency:
    - A. Viscosity, Furol, A. S. T. M. standard method D 88-30.
        - *a.* At 77° F. for dust layers, primers, and all products which are applied without warming.
        - *b.* At 122° F. for all products, including cut-backs, which are warmed slightly before application.
        - *c.* At 210° F. for all highly viscous products which must be heated to approximately 200° F. or above before application and which have a viscosity of less than 300 seconds at this temperature.
    - B. Float test at 122° F. on materials having a viscosity, Furol, of more than 300 seconds at 210° F. A. S. T. M. standard method D 139-27.
3. Distillation—for all products:

    Total distillate, by volume, at the following temperatures: 437° F., 600° F., 680° F.

    A. S. T. M. standard method D 20-30 with the following modifications:

    Sample distilled shall be 200 cubic centimeters, the weight of this volume to be calculated from the specific gravity at 60° F.

    Bulb of thermometer shall be immersed to a point one-fourth inch above bottom of flask.

    Condenser shall be water cooled.

    Distillate shall be collected in 100-cubic-centimeter graduated glass cylinders and the amount of distillate shall be reported in percentages by volume of water-free material.

    Distillation shall be stopped at 680 °F. and the amount of distillate read to the nearest cubic centimeter after the condenser has been allowed to drain thoroughly into the receiver. The total residue shall then be poured immediately into an 8-ounce tin and allowed to air cool without covering to a temperature below its fuming point suitable for pouring. It shall then be stirred and poured into proper receptacles for additional tests.

    Temperatures observed in the distillation test shall be corrected for the effect of the altitude of the laboratory in which the test is made. (See A. S. T. M. standard method D 86-30.)
4. Tests on residue from distillation:
    - *a.* Float test at 122° F. on all residues having a float of more than 25 seconds at this temperature and having a penetration of more than 300 at 77° F. (100 grams, 5 seconds.) A. S. T. M. standard method D 139-27.
    - *b* Penetration at 77° F. on all residues of less than 300 penetration (100 grams, 5 seconds). A. S. T. M. standard method D 5-25.
    - *c.* Ductility at 77° F. on all residues of less than 200 penetration at 77° F. (100 grams, 5 seconds). A. S. T. M. tentative method D 113-26T.
    - *d.* Solubility in carbon disulphide—all residues. A. S. T. M. standard method D 4-27.

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[2] Method of test for flash point with Tagliabue open cup: The Tagliabue open cup flash tester shall be used. The instrument shall set firm and level.

Fill the metal bath cup with water having a temperature at least 20° F. below the probable flash point of the oil to be tested, leaving room for displacement by the glass oil cup, which is then placed in the bath.

Fill the glass oil cup with the oil to be tested to within five-sixteenths inch of its upper level edge. See that there is no oil on the outside of the cup, or upon its upper level edge, using soft paper to clean the cup.

Adjust the horizontal flashing-taper guide-wire in place. Suspend the thermometer, with its bulb well covered by the oil. Heat bath with small flame lamp having the flame so adjusted that it will raise the temperature of the oil at a rate of 2½° F. per minute, without removing the lamp during the whole operation.

For viscous liquids it is necessary that the liquid be stirred at intervals during the test.

Remove air bubbles, if any, from the surface of the oil before first trial for flash is made.

At the proper trial temperatures, noted below, try for flash with a small bead of flame (not over one-eighth inch) by drawing it quickly and without pause across the guide wire from left to right.

The temperature at which the first or initial flash is obtained is called the flash point.

Trial temperatures: For materials which may be expected to flash at about 80° F., try for flash at 70° F., then at 75°, 77°, 79°, 81°, 83°, and 85°. For other materials try for flash first at a temperature about 20° F. below the expected flash point and then try for flash at every 5° F.

MUCH PROGRESS MADE AT REGIONAL MEETINGS

The discussions which took place at the second series of regional meetings related entirely to the adoption or elimination of test requirements and no attempt was made to set up test limits for specification purposes. The action taken at the meetings was quite informal in the sense that the acceptance or rejection of a test by a State was considered to represent the intent of that particular State with respect to its own specifications and the action of a majority was not considered as binding on the minority.

The important accomplishments of these meetings, which were attended by representatives of 30 of the 48 State highway departments, are summarized as follows:

The adoption of the tests for flash point, as given in the simplified scheme, was practically unanimous, as was also the adoption of the Saybolt-Furol viscosity test to replace the Engler test. With few exceptions it was agreed that the temperatures at which viscosity determinations would be made would be 77° F., 122° F., and 210° F. It was also generally agreed that the float test at 122° F. would be used for materials having a viscosity, Furol, of more than 300 seconds at 210° F.

With two exceptions, the distillation test as described was adopted for cut-back materials by all the States represented, but the majority were not prepared at present to extend its use to materials other than cut-backs. A number announced the intention of making a further study of its suitability for the materials not classed as cut-backs. Discussion of this question led the representatives of the States in region No. 3 to define cut-backs as follows: Cut-back asphalt is a liquid asphaltic product which, when distilled to 680° F., yields a residue of less than 300 penetration at 77° F.

It was generally agreed that the end point of the distillation test should be 680° F. and that the volume of distillate should also be determined at 437° F. and 600° F. One region agreed to use no other temperatures. One region added another cut at 374° F.; another made the use of this temperature optional; and the other two regions added to it the temperatures of 302° F., 320° F., and 500° F., with the general agreement that not more than four temperatures in addition to that of 680° F. would be used in any one specification.

The agreement to adopt the proposed tests of residues from the distillation test was practically unanimous.

With respect to the elimination of tests not included in the simplified scheme of analysis, there was less unanimity of opinion than in the case of the tests included in the scheme, but, in spite of this, considerable progress was made.

As has been noted, the agreement to replace the Engler with the Saybolt-Furol viscosimeter was practically unanimous and only a small minority expressed the intention of retaining the test for insolubility in naphtha. However, a large majority favored the retention of the test for the determination of asphalt content, particularly for materials other than cut-backs, until such time as it is more definitely established that the tests in the simplified scheme are adequate to replace it. A large number of those who favored the continued use of the asphalt content test also favored a ductility test on the residue.

About half the States represented at these meetings will continue to use the volatilization test (loss on heating for five hours at 325° F.) for the time being, at least, although of these about half will use it only for materials other than cut-backs. The States retaining the volatilization test generally will retain also penetration or float tests on the residues from volatilization.

The volatilization test at 212° F., which has been used to a limited extent in the past, was rejected with only one dissenting vote. The elimination of the test for fixed carbon was also practically unanimous.

The test for the determination of paraffine scale, concerning which there has been much discussion during the past few years, particularly in the States of region No. 5, received, in the various meetings, only one vote for its retention. It is known, however, that at least one other State, which was not represented at the meetings, will continue to use the test in spite of its questionable accuracy and problematical value.

The test for solubility of original materials in carbon tetrachloride will not be continued to any extent except in one region.

The vote for the elimination of specific-gravity determinations on original materials and distillates and float tests on original materials at 77° F. and 90° F. was so nearly unanimous that their continued use will be practically negligible.

This summary of accomplishment evidences a truly encouraging progress in the simplification program, particularly when one considers the relatively short time during which the cooperative effort has been carried on. However, much remains to be accomplished before the program can be brought to a satisfactory working basis. Misunderstandings and misconceptions must be cleared up and a more complete knowledge and understanding of many details must be obtained. As an aid in this direction it is believed that a discussion of the logic of the proposed simplified scheme of testing and a critical analysis of other tests still in common use in certain regions will be of service.

ESSENTIAL PROPERTIES TO BE DEFINED IN SPECIFICATIONS FOR LIQUID ASPHALTIC PRODUCTS

Without regard to the individual tests which may be used for the purpose, it seems important to specify certain fundamental properties of liquid asphaltic products, depending upon their method of use and the purpose which they are expected to serve. Of these, the most important appear to be:

1. Original consistency.
2. Ultimate consistency as representing the material developed after application.
3. Relative rapidity of the change, if any, from original consistency.
4. Amount and character of active bitumen remaining after application.

Original consistency.—The determination of consistency of the original material is essential and, in the case of liquid asphaltic products, it is commonly ascertained by a test for viscosity. Some form of viscosity test almost invariably has been incorporated in specifications for materials of this character. The method chosen for determining viscosity of highway materials usually has involved the use either of the Engler viscosimeter or of the Saybolt-Furol viscosimeter, the test being made with each instrument at a variety of temperatures ranging from 77° F. to 212° F.

As has been noted, the decision to substitute the Saybolt-Furol test for the Engler test in the simplified scheme of analysis was practically unanimous and it was generally agreed to make the viscosity determinations at temperatures of 77° F., 122° F., and 210° F. The primary reason for the selection of the Saybolt-Furol instrument in preference to the Engler is the considerably shorter time which is required for the test. The availability of apparatus with multiple tubes and an accurate, automatic temperature control are added advantages of the Saybolt-Furol test.

The maximum temperature originally proposed for the viscosity test was 212° F. (100° C.), but this was changed to 210° F. to eliminate the general need for the use of an oil bath instead of a water bath. The difference between the two temperatures is not particularly important, but the advantage, if any, gained by the change to 210° F. is not general, since in laboratories located at altitudes much over 1,000 feet above sea level the oil bath must be used for either temperature.

The viscosity, or resistance to flow, of all liquid bituminous materials may be decreased by raising their temperatures, but the decrease in viscosity per unit rise in temperature often varies greatly for different products. Therefore, a given viscosity value at an elevated temperature, such as 212° F., by no means indicates the viscosity or consistency of the product at a more normal temperature.

Consequently, it is desirable when possible to ascertain the consistency of bituminous road materials at what may be considered a normal temperature. Since 77° F. has been accepted generally as an average normal temperature, it is desirable to make the viscosity test at this temperature when it is possible to do so satisfactorily. However, many of the liquid asphaltic products are so highly viscous that a satisfactory determination at 77° F. can not be made. In such cases it is necessary to make the test at some elevated temperature, and this preferably should be within the range to which the material may be subjected in service subsequent to its application. A temperature of 140° F. represents the approximate maximum of this range. Many materials which are too viscous to be tested at 77° F. may be tested accurately at 122° F., and this test temperature, which is within the service range, has come into quite general use for such products. In the proposed simplified scheme of analysis, therefore, 77° F. and 122° F. were put forward as being satisfactory temperatures for the great bulk of products now in use.

Some products so nearly approach the semisolid state in original consistency at normal temperatures that satisfactory viscosity tests can not be made on them even at a temperature of 140° F. or considerably higher. For such products the temperature of 210° F. has been adopted. As a result of further consideration of the matter and subsequent analysis of the cooperative test data, it now appears that it might be advantageous to eliminate determinations of viscosity at this high temperature and substitute the float test at 122° F. as a measure of consistency of those products which can not be tested for viscosity at 122° F.

It seems highly desirable to control the consistency of all materials by tests at temperatures within the range of those encountered in service subsequent to application. At the present time the float test appears to be the only acceptable test which is satisfactory for such determinations when the material is too viscous for a viscosity test and too soft for a penetration test within this range of temperature. While it can not be translated accurately into terms of viscosity or penetration for all types and grades of asphaltic material, nevertheless it gives an idea of approximate consistency which is far more significant than a viscosity test at 210° F.

As has been stated previously, the viscosity at 210° F. or 212° F. does not indicate consistency at normal temperature, and from the standpoint of the application of a material no record of viscosity at such an elevated temperature appears necessary. For all practical purposes, therefore, it is believed that the float test at 122° F. will be satisfactory as a measure of consistency of the class of materials which are too viscous for a viscosity test at 122° F. and too soft for a penetration test at 77° F.

Ultimate consistency as representing material developed after application.—In certain types of highway construction and treatment, liquid asphaltic materials are used in preference to the semisolid varieties to facilitate their application to or manipulation with the nonbituminous constituents of the roadway surface. If it were practicable to use a much more viscous product, such a product frequently would be used preferentially to produce without delay the results which it planned to secure ultimately. In other words, while not invariably so, it is frequently desirable that the viscosity or consistency of the original material should become greater after it has been applied. Therefore, it is desirable to ascertain and properly describe in specifications the approximate degree to which the original material will harden or increase in viscosity after use. Various tests have been used with this general purpose in mind, but comparatively few actually give the desired information for reasons which will be discussed later. Before the simplified scheme of analysis was proposed, these tests were carefully considered from all standpoints and it was concluded that the distillation test, with the thermometer bulb in the liquid rather than in the vapor phase, came nearer to giving the desired information than any other single test.

A distillation test carried to an end temperature of 680° F. has been used for some years by a number of the State highway departments in the testing of cut-back asphalts. The definition of a cut-back, which consists of an asphalt cement fluxed to fluid consistency with a volatile distillate, has been given. Such products, when exposed in relatively thin films, are known to revert to the approximate consistency or penetration of the original asphalt cement through loss by volatilization of the solvent. In order to remove the solvent rapidly by distillation it is necessary to carry the distillation to 680° F. In the case of cut-back asphalts, therefore, in order to determine the approximate consistency which the original material may be expected to attain, after application, the material is distilled to 680° F. and the penetration of the residue remaining after distillation is taken as an indication of the increase in consistency which may be expected to occur under conditions of use.

Since cut-back products tend to harden through the loss of volatile constituents which are removed as distillates in the distillation test, it seems reasonable to believe that the test is as applicable to the liquid asphaltic products of other types, and that a determination of consistency of the residue obtained from distillation will serve as a reasonably accurate measure of the degree of hardening which any such product may be expected to undergo after application.

The penetration test at 77° F. is a satisfactory measure of consistency where it can be used, but many of the liquid asphaltic products which have proved adequate for certain purposes will not yield a distillation residue sufficiently hard to be subjected to the penetration test. For such residues, therefore, the float test at 122° F. has been included in the simplified scheme of analysis. This permits the rating of consistency of residues as liquid if the float test is less than 25 seconds; by float test if this exceeds 25 seconds and the penetration is more than 300; and by the penetration test if the penetration is less than 300.

Rate of change in consistency.—In many types of construction the rate of change in consistency after use is important and it is desirable to have a knowledge of the probable rapidity with which this change will take place and also to have a means for its approximate control. Here again the distillation test proves superior to other tests which have been used, since it permits the determination of the percentage of volatile materials which distill off at temperatures intermediate between the initial boiling point and the final test temperature of 680° F. The value of the distillation test in this connection has already been demonstrated in its application to cut-back asphalts. By the proper selection of temperatures and corresponding percentages of distillate the rate of change in consistency can be controlled within reasonable and practicable limits. Probably not more than three temperatures at properly spaced intervals will be required to give the necessary information. In the simplified scheme of analysis the temperatures which have been proposed are 437° F., 600° F., and the end temperature of 680° F. These temperatures should be sufficient for road oils and slowly hardening cut-backs, but for quickly hardening cut-backs an additional lower temperature, possibly 374° F., may be required.

Amount and character of active bitumen remaining after application.—In addition to obtaining the change in consistency of a material by measuring the consistency of the residue from the distillation test and ascertaining the relative rapidity of the hardening process, the distillation test also makes it possible, by recording the volume of residue, to determine the amount or percentage of the final product in terms of the original material.

Other characteristics of the residual bitumen, in addition to its amount and consistency, are of interest. Thus, its freedom from admixture with foreign substances can be readily obtained by the test for solubility in carbon disulphide, which has been included in the proposed scheme of analysis. It is suggested that the test for solubility in carbon tetrachloride may be even more useful since it is indicative of all the information furnished by the carbon disulphide test and, at the same time, it may assist in differentiating between materials produced by straight distillation or mild cracking and those produced by severe cracking processes, which may have characteristics approaching those of tar.

For residues which would be classed as asphalt cements and which are sufficiently hard to have a penetration of less than 200 at 77° F. it was considered desirable to include, in the scheme of analysis, a test for ductility of the residue at 77° F.

FLASH POINT TEST ESSENTIAL AS SAFETY MEASURE

In addition to what are believed to be essential tests, one other test may legitimately be included in the simplified scheme of analysis, namely, the flash-point test.

Certain classes of asphaltic road materials containing a very volatile distillate may flash at relatively low temperatures. The regulations of the Interstate Commerce Commission governing the rail shipments of such materials require that the flash point be determined with the Tagliabue open cup and that materials which flash at a temperature of 80° F. or less be identified by a special label. Accordingly, the use of the Tagliabue open cup has been specified in the simplified scheme of analysis for materials of low flash point and the method of test approved by the Bureau of Explosives has been adopted.

In addition to a flash point test to determine compliance with shipping regulations, it is desirable, from the standpoint of safety, to limit the minimum flash point of materials which are to be heated before application. For materials flashing at 175° F. or higher the test with the Cleveland open cup (A. S. T. M. standard method D 92–24) has been adopted. Since this method restricts the test to a minimum temperature of 175° F., this temperature has been made the upper limit for flash point determinations with the Tagliabue instrument.

TESTS OF QUESTIONABLE VALUE

We have discussed the tests included in the simplified scheme which are believed to be essential for defining desirable properties of liquid asphaltic products for highway use. A critical examination of other tests which are commonly employed in the specifications for such products is in order. In the 1932 series of regional conferences quite general agreement was reached by the representatives of the State highway departments regarding the advisability of eliminating such tests as specific viscosity (Engler), specific gravity, fixed carbon, paraffine scale, and float test on original material at other than 122° F., and therefore further discussion of these tests is unnecessary.

Tests for asphalt content or residue of a given penetration.—Probably no single test has become more firmly established in specifications for liquid asphaltic products than that for the determination of percentage of residue of a given penetration, commonly known as the asphalt content of the product. A general feeling seems to prevail on the part of highway chemists and engineers not only that the amount of such residue indicates the degree of asphaltic character possessed by the material, but also that in some way the actual percentage of asphalt content measures the serviceability of such products for highway purposes. While no direct claim is made that if a product develops a given percentage of asphalt content by this test, it will develop such asphalt in place after use, the feeling prevails that in some way or other such is the case, and so the liquid asphaltic products frequently are graded on the basis of their so-called asphalt content.

The test is made by maintaining a weighed sample exposed to the air in an open receptacle at a temperature of 480° to 500° F. for whatever period of time is required to reduce it to the specified penetration, usually either 100 or 80 penetration at 77° F. No experimental data have ever been made available to show that the residue so obtained is even approximately reproduced when the material is subjected to service conditions, while there are excellent reasons which indicate that such can not be the case except, perhaps, for cutback asphalts. However, the test is not generally applied to cut-backs, since the distillation test has come to be recognized as giving much more accurate information relative to the probable behavior of such materials after application.

The reasons why the asphalt content test does not give information as to the character of material developed after application are as follows:

(1) It is a well-known fact that asphaltic materials, when subjected to the temperatures at which this test

is conducted, are, as a rule, highly susceptible to oxidation, and that oxidation over the period of time required for the test produces chemical changes which are not produced at the normal temperatures encountered under ordinary service conditions. These chemical reactions result in an artificial hardening of the product, so that a residue which does not represent an actual residual constituent of the original material is manufactured by the test itself.

(2) Even if the residue of given penetration actually existed in the original material, no service conditions would be sufficiently severe to develop this residue in place except possibly in a thin film on the highway surface. Thus, many liquid residual petroleums, from which a given percentage of asphalt of a given penetration may be obtained from distillation processes as conducted in a refinery, will never develop this asphalt as such under service conditions because the oils which hold the actual asphalt in solution are nonvolatile at ordinary atmospheric temperatures and never leave the asphalt residue under conditions of use.

In view of these facts, the question at once arises as to what purpose an asphalt residue serves, even when present, if it is never allowed to develop as an asphalt cement. Certainly it can not develop high binding value because no liquid asphaltic product which does not harden can be shown to possess high binding properties any more than can any permanently liquid glue. Hardening is necessary to develop binding properties and it is therefore of far greater importance to determine the probable change in consistency or hardening of the material after use than it is to determine the amount of material of a given hardness which can be obtained only by artificial means which are not duplicated by conditions of service.

It has already been pointed out that the distillation test is capable of indicating the probable degree of hardening that the material may be expected to reach after application to or incorporation with the mineral aggregate. However, quite a few engineers and chemists, who have come to depend upon the asphalt content test, feel that the distillation test does not always give them the information which is desired because of the fact that many liquid asphaltic products, which will yield a relatively high asphalt content, often show little loss by distillation to 680° F. and yield a distillation residue which is still fluid. It has been pointed out that if, upon distillation, a liquid residue remains, the product can not be expected to develop an asphalt cement after application. So long as the asphalt cement which may exist is held permanently in solution, it appears to be quite immaterial what the exact percentage of such asphalt cement may be.

It would seem, therefore, that the asphalt content test might well be abandoned altogether for use in specifications, together with such tests on the resulting residue as are sometimes specified. In this connection, particular reference is made to requirements for ductility of residue of a given penetration since the physical properties of the residue will vary, depending upon the temperature at which the residue is produced and the length of time it is exposed to the oxidizing action of air at that temperature.

Volatilization loss at 325° F. and 212° F. and tests on residues.—Determination of loss by volatilization at 325° F. and consistency of the resulting residue are tests which originally were devised for asphalt cements and fluxes in connection with their use in hot asphalt paving mixtures. In the preparation of such mixtures the asphaltic constituents are customarily maintained at a temperature of approximately 325° F. over rather long periods of time and the mineral constituents of the mixture are also heated to approximately this temperature. Therefore, it is quite logical to make the tests on these products in connection with such use for the purpose of determining loss by volatilization and the degree of hardening that may be expected to occur during manipulation.

As the use of the liquid asphaltic products was developed for low-cost road construction, it became rather customary to utilize the volatilization test at 325° F. in specifying such products, merely because it had been used extensively in connection with materials for hot-laid asphalt pavements. Apparently no thought was given to the fact that many of these liquid products were never heated to the temperature at which the test was run and were, in most cases, applied to or incorporated with unheated mineral aggregate. In reality, therefore, there is little excuse for applying this test to liquid asphaltic products except when it is intended that they shall be used as permanent fluxes for harder asphalts.

It is true that, in the case of products which may be expected to increase in consistency or harden after use, appreciable quantities of volatile constituents are driven off during the test and that a determination of the consistency of residue will indicate some degree of hardening. However, the test is run at the maximum temperature for such a long period of time that it does not in any way reproduce service conditions. At best, no indication of rapidity of hardening is given by the test results, such as is very definitely shown in the distillation test by the percentage of distillate obtained at various temperatures between the initial boiling point and the end-point of distillation. This fact has been recognized by some testing engineers who accordingly have utilized a volatilization test at 212° F. in addition to the test at 325° F. The relative difference in loss and in consistency of residues from the tests at these two temperatures tend to establish some relation between the rate of hardening of different products, but here again the information is not nearly as complete or reliable as that given by the distillation test. Therefore, it appears that all of the tests for loss by volatilization and tests on the consistency of the residues so produced could well be abandoned in favor of the distillation test which, at least for cut-backs, has demonstrated its direct relation to the behavior of the material after application.

Bitumen insoluble in naphtha.—In spite of the very considerable amount of study which has been devoted to the subject, the chemical composition of asphalts is but little understood. Their predominating constituent is bitumen, which is completely soluble in carbon disulphide. By treating an asphalt with a naphtha of low boiling point it is possible to precipitate a portion of the bitumen in the form of a dark brown or black powder which may be separated by filtration. These insoluble constituents have been called asphaltenes. Asphaltenes are complicated mixtures of hydrocarbons, the exact function of which, in an asphalt, is not thoroughly understood. All asphalts contain such constituents, but the percentage varies greatly with the character of the naphtha, such as its gravity and range of boiling points.

When asphalts from only a few individual sources were commercially available for highway work and when asphaltenes in these particular asphalts were found to lie within fairly narrow quantitative limits, it became customary to include in specifications the test for bitumen insoluble in naphtha. The determination itself served as no direct insurance of quality of the product but rather as a descriptive test.

Later, as asphalts were developed from numerous other sources, it became apparent that the percentage of asphaltenes which they contained varied within wide limits but that such variation had no direct traceable relationship to the relative behavior or serviceability of these products. Moreover, it was learned that the percentage of asphaltenes in an asphalt obtained from a given source could be increased materially by subjecting the material to oxidation by the blowing process. Since no definite relationship between desirable properties of an asphalt and its asphaltene content could be established, the test was gradually eliminated from specifications for asphalt cements although, for no apparent good reason, it is still retained to a limited extent in certain specifications.

Its application to liquid asphaltic products, however, has become quite general in certain sections of the country due to an opinion that in some way the percentage of asphaltenes indicates the asphaltic character of the product. It is true that those petroleum products which contain but little asphalt in solution show a very low percentage of asphaltenes, but, because of the tremendous variation in percentage of asphaltenes in satisfactory asphalts from different sources, it is apparent that wide variations will occur in liquid products which contain appreciable quantities of these asphalts in solution. The percentage present in a given product therefore can hardly be expected to indicate the suitability of the product for a given use.

Many of the State highway departments have agreed to the elimination of this test from specifications, and it is hoped that unanimous agreement upon elimination will ultimately be reached. At best, the test is exceedingly inaccurate and subject to wide variations in the results obtained by different operators. This is due not only to the difficulty in standardization of the naphtha solvent but also to the fact that the solvent is constantly changing in composition through partial volatilization during the filtration process and the further fact that there is no clear end point for separation of soluble from insoluble material. The filtrate shows discoloration even after prolonged washing of the precipitated asphaltenes on the filter bed. Probably no other test which has been applied to asphaltic products can be less easily checked by different operators than the test for bitumen insoluble in naphtha, and this fact alone should be sufficient for its elimination from specifications.

Test requirements for original components of blended or cut-back products.—In specifications for blended products such as cut-back asphalts some State highway departments, in addition to the requirements for the finished product, include also detailed requirements both for the original asphalt cement and the original distillate to be used in the manufacture of the product. Such practice is believed to be unnecessary, since it is doubtful if it constitutes a better guaranty of quality than do properly drawn specifications for the finished product only. It may also be uneconomical since it tends to hamper the manufacturer and may eliminate desirable competition.

If specifications for a cut-back product cover its original consistency, the consistency and character of its residue from the distillation test, and the rapidity with which it may be expected to develop this residue after application, it is immaterial just how it has been manufactured. Practically the same finished product often may be produced from a variety of individual constituents and this fact is clearly recognized in specifications for other materials, as for example, Portland cement, in which no attempt is made to specify the exact nature of the original argillaceous and calcareous constituents from which it is manufactured.

In the petroleum industry the production of distillates is of major importance and the manufacture of cut-back asphalts constitutes a very small part of the business. In the production of these distillates, crude oils from a great many sources are refined by a variety of processes and probably no two refineries employ exactly the same refining processes on the same crude oils. Moreover, their markets for distillate products vary considerably, so that all do not produce the same standard distillates. Before many of these distillates are placed on the market they are subjected to expensive treatments to improve their color and odor, which characteristics are immaterial from the standpoint of use in the manufacture of cut-back asphalts. Unfinished distillates, therefore, are often the most economical to use in such highway materials, and it would often be impractical to rearrange refinery practice to produce a special specification distillate for the relatively limited use it would have for such products.

Paving asphalts, on the other hand, have become well standardized products manufactured in a definite number of grades ranging from very hard to very soft. In general, the only difference between these grades is the extent to which the original crude petroleum, from which they are obtained, has been distilled. In other words, the harder varieties may be produced from the softer ones by distilling off a small amount of oil from the latter. It follows therefore that the softer grades may be produced from the harder by fluxing the latter with a high-boiling distillate oil similar to that which was removed in reducing them from the softer consistency to their original degree of hardness.

Many unfinished refinery distillates available for producing cut-back asphalts contain a small amount of high-boiling oils similar to those which would be removed in converting a soft asphalt to one of harder consistency. When present in a cut-back asphalt this portion of the original refinery distillate remains in the residue from the distillation test and, therefore, serves as a permanent flux for the asphalt cement used in the preparation of the cut-back. The asphalt cement recovered from the distillation test is, therefore, of softer consistency than the asphalt originally used.

Specifications for the finished product require that the residue from the distillation test shall be within certain limits of consistency which are considered essential from the standpoint of behavior after use. Provided this requirement is met, it is apparent that, so far as the consumer is concerned, it is immaterial whether the original cut-back is made with a relatively soft asphalt fluxed with a distillate which will be completely removed by the distillation test, or with a relatively hard asphalt fluxed with a distillate containing a high-boiling fraction which is not removed by the distillation test but which permanently reduces the hard asphalt to the consistency specified for the residue.

These relations, which are of great importance to the manufacturer, are clearly illustrated in the diagram of

(Continued on p. 104)

THE BATCHING PLANT IN CONCRETE PAVING WORK

By ANDREW P. ANDERSON, Highway Engineer, Division of Management, United States Bureau of Public Roads

THE BATCHING plant, the primary purpose of which is to supply the hauling units with the materials which they are to carry to the concrete mixer, is an essential part of the auxiliary equipment necessary to the efficient functioning of the customary portable paving outfit. In these outfits the paver or mixer is the key producer with which all equipment, operations and procedures must be properly coordinated in order to produce high operating efficiencies and low unit costs. This is due to the fact that the rate at which such a paving outfit can produce square yards or cubic yards of finished concrete pavement is definitely fixed, first, by the time required to pass the specified batch through the mixer, and, second, by the extent to which this operation can be maintained with clocklike precision as a continuous performance. It is this factor of maintaining steady, continuous operation which is so difficult.

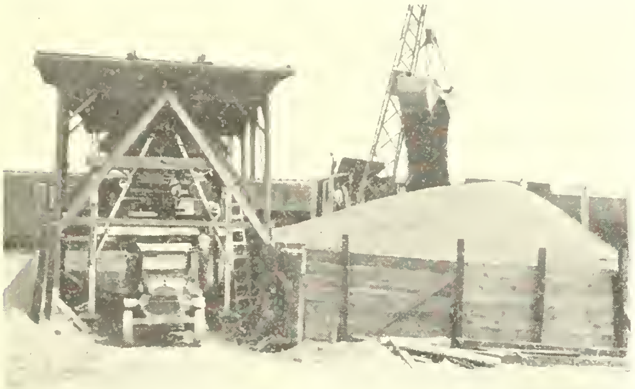


Figure 1.—A Three-Compartment Bin and the Effective Use of Planks Greatly Reduces the Amount of Space Necessary to Handle Two Sizes of Aggregate

While it is true that no matter how large or how well the outfit is equipped, organized, and managed, the rate fixed by the characteristics and requirement of the mixer can not be exceeded, it is equally true that no necessary or prescribed operation or requirement is so trivial or so insignificant that it can not delay or hold up production just as effectively as though for the time it were the sole essential factor. Every piece of equipment, every group or assembly, must therefore be able to perform its required function with the regularity and within the limits set by the mixer; otherwise a loss in production is the unescapable penalty. This applies with particular emphasis to the batching plant through which all the principal raw materials must pass.

The three essential functions of the batching plant or loading yard are: (1) To receive the incoming stone or gravel, sand and cement, and provide for these such storage as may be necessary or desirable to iron out the fluctuations of the incoming supply so that the continuous demands of the mixer may be met; (2) to form or combine these materials into unit quantities of the size and the proportion required for the batch at least at the rate and with the regularity demanded by the mixer; and (3) to provide for the transfer of these batches or unit quantities into the hauling units with the least possible delay or interruption to the primary function of the trucks, which is to transport the batched materials from the loading yard to the mixer.

The usual batching plant for supplying a 27E paver capable of producing at a maximum rate of from 45 to 55 batches, or from 50 to 70 cubic yards of concrete per hour, consists essentially of a crane with a 1 or 1¼ yard bucket, one or two bins equipped with batch-weighing devices, and a cement loading platform or a cement bin, all generally located at a railroad siding where there is room for ample stockpiles; and if sack cement is used there must usually also be room for a cement shed with a capacity of at least one day's supply.

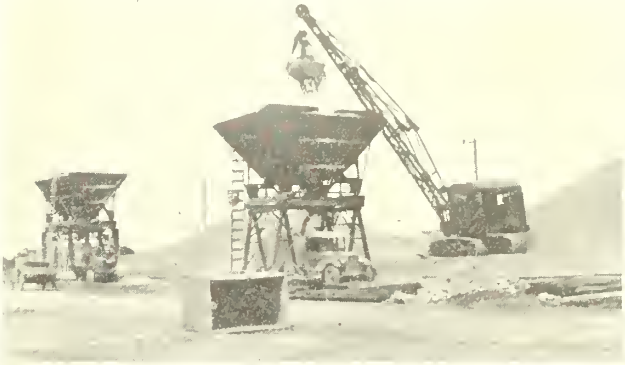


Figure 2.—Two Bins With Straight Drive Through Both. Stock Piles on Opposite Sides of Track. Crane Working on a Ramp Made of Coarse Aggregate

The function of the crane is to unload materials from cars to bins and to stockpiles and in the absence of cars to supply the bins from the stockpiles. It should in general have a capacity of not less than 1 or 1¼ cubic yards and be capable of maintaining a regular cycle of not to exceed 25 seconds per load from stockpile to bin when working on an angle of about 90°. From cars to bins the crane should be able to maintain an average cycle of not to exceed 30 seconds. Such a crane should easily be able to keep up with the mixer even at maximum production if permitted to work the coarse aggregate from cars to bins.

Table 1 shows the average rate of operation, on 10 jobs, of cranes in good condition, equipped with 1¼-yard buckets.

Table 1.—*Crane operation, unloading cars to bins—Averages for 10 jobs—1¼-yard bucket cranes, in good mechanical condition*

	Sand	Gravel
Load bucket, seconds	7.8	8.3
Swing, seconds	8.0	8.4
Dump, seconds	2.2	2.1
Return swing, seconds	7.6	7.9
Average cycle, seconds	25.6	26.7
Average angle of swing, degrees	100	112
Average load per bucket, cubic yards	.88	.76
Average number buckets per cubic yard	1.136	1.316

Average time cranes worked, per cent, 66.

If all of the materials must first go to the stockpiles, some night unloading of cars to stockpiles will probably be required when mixer production is high. Where ground conditions permit, the speed of crane operation can be increased considerably by working on a runway about 6 or 7 feet high so that the operator can see into the cars. Such a ramp and runway can be built up from coarse aggregate and used as a crane run until the last day or so of paving, when the material in the ramp is utilized as a stockpile.

Occasionally the siding is so situated that one or more pits can readily be located beneath and to the yard side of the track. The materials, shipped in dump-bottom cars, are then dumped into these pits to be taken by either a belt conveyor or the crane to the bins or stockpiles as required. This method saves some time in that the crane is not delayed by cleaning out the bottoms of the cars or in spotting new cars.

If the coarse aggregate is of two sizes, a 3-compartment bin should be used. Two bins will generally add from one-half to three-quarters minute per batch to the time constant of the trucks aside from increasing the chance for other delays to occur. The 3-compartment bin also has the advantage that it can generally be loaded by the crane more readily than the two bins where the layout is properly made and some convenient form of power unit is provided for moving the cars. Moving the crane from one bin to the other usually consumes more time than is lost in keeping three cars within reach for loading into a 3-compartment bin. The capacity of any bin or set of bins should at least be sufficient for one hour's run of the mixer. At least this much ready storage is necessary to absorb the probable minor crane delays and interruptions which might otherwise result in somewhat similar delays to the mixer. It is much cheaper to provide this amount of reserve storage in bins than in trucks.

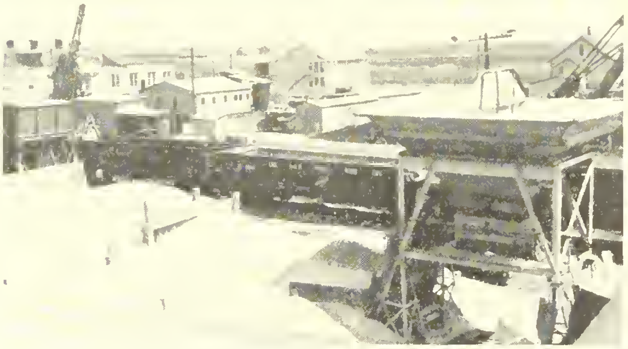


Figure 3.—Lack of Room for Ample Stock Piles and Specifications Requiring Three Sizes of Coarse Aggregate Necessitated the Use of Two Cranes

In the case of the cement, conditions are sometimes such that the cement bags are loaded directly from the car; but working wholly from within the car is difficult and usually it is preferable to construct a platform so that the bags can be trucked from the car and dumped directly from this platform on to the truck. For this purpose the layout must usually be studied rather carefully in order to place the cement platform so that the bags can readily be dumped from the hand trucks directly on to the waiting truck without any rehandling or moving around. To make this possible the platform must be about 6 inches higher than the truck body and so located that the trucks can readily drive very close along the side. Depressing the driveway is sometimes resorted to, but if the soil is heavy and the rainfall abundant this method is likely to prove a disappointment. The alternative is tossing the bags, one at a time, on to the truck by hand. This ordinarily requires from 10 to 15 man-seconds per bag.

The most common way of handling bulk cement at the present is in 2-wheeled buggies. These may be of either the end or bottom dumping type. This method requires a platform which should be long enough so that the unloading can proceed simultaneously from two railroad cars. This gives more room for the men to work and so increases their efficiency. For best results that part of the buggy track which extends from the platform out over the truck should be hinged at the platform edge and provided with a counter weight so that the push of the advancing buggy wheels will cause it to lower to a horizontal position but immediately return to about 75° or 80° with the horizontal as soon as the buggy has been dumped and withdrawn. A canvas boot long enough to reach the aggregate in the truck body or to reach well into the special cement container should be provided. Either the canvas of the boot or some other material should extend high enough above the boot to prevent a strong wind from whipping up the cement as it is being dumped. Table 2 gives the average results of a series of stop-watch studies of handling bulk cement in buggies.

Table 2.—*Handling bulk cement in hand buggies—All time readings in utilized man-seconds—630 pounds of cement to each buggy*

	Job 1	Job 2
Load buggy	120.7	134.7
Wheel buggy to scales	15.9	17.2
Weigh and adjust to proper weight	16.4	17.8
Wheel buggy to dumping place	10.4	10.3
Dump cement into truck	17.8	14.7
Return buggy to loaders	11.3	13.6
Total utilized man-seconds	192.5	208.3

In hot weather, loading the buggies is very trying labor. The number of shovelers must therefore be sufficient to afford frequent rest periods. On fast jobs a total of six men are usually employed on the cement in addition to the weight inspector.

The time constant of the trucks—i. e., the average total time spent by the truck during each round trip in other activities than the actual hauling of the load to the mixer and in returning again to the yard—is generally increased somewhat when bulk cement is used, the amount depending on the methods of handling the cement. Three rather general methods are in use in various sections: (1) Dumping the cement in between the aggregates, as, for example, coarse stone, sand, cement, and then second-size stone; (2) dumping cement on top of the batch and covering with a canvas or tarpaulin; and (3) carrying the cement in a special container on the truck.

Of these methods the first is generally the fastest wherever two aggregate bins are used. The cement platform or cement bin is then located between the two aggregate bins. Taking on the cement thus requires no more time than when using bag cement. The second method usually requires from 15 to 30 seconds to roll back and tie down the tarpaulin. A variation of this method is sometimes met with in which the cement is covered with a thin layer of sand or aggregate instead of with a tarpaulin. The usual procedure is to smooth off the top of the load throwing the material in a heap along the edge so that when the cement has been dumped this material is available for covering. This method usually requires two men and a delay of from one-half to one minute for each single-batch truck and about twice that amount for a 3-batch truck. The special containers used for carrying bulk cement are far from standardized. The time required to load them therefore varies with different types. In general, about 25 to 45 seconds are required for loading the cement

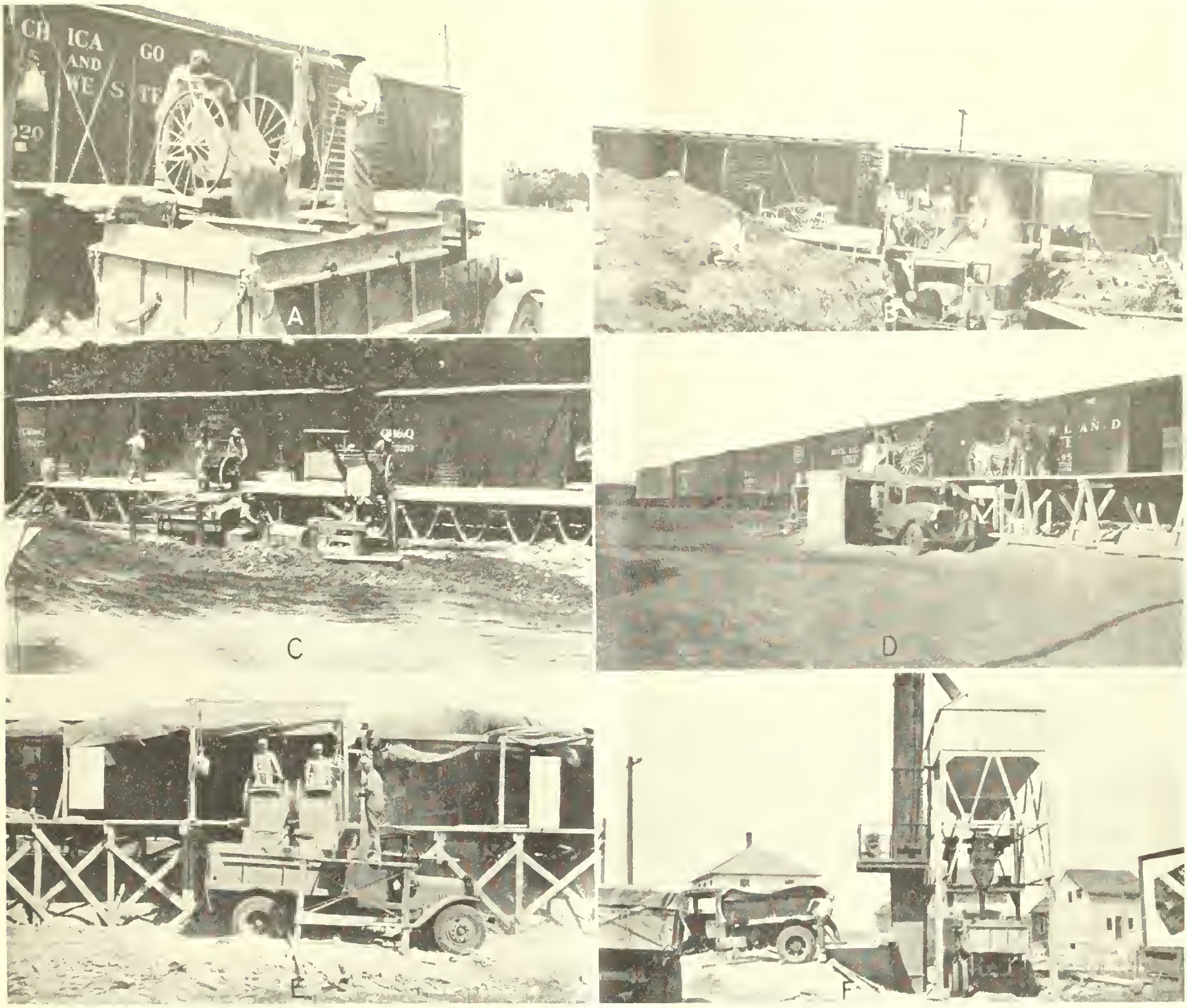


Figure 4.—Handling Bulk Cement:

A.—Things to be Avoided in Handling Bulk Cement: Platform Only Long Enough For 1 Car; No Boot to Prevent Blowing or Spilling of Cement; 3-Batch Trucks But Room Only to Dump 1 Buggy at a Time

B.—Ground Conditions are Sometimes Such That a Convenient Layout is Impossible

C.—The Loading Platform for Bulk Cement Should Always be Long Enough to Permit Work in 2 Cars

D.—A Workmanlike Cement Platform, Which Permits Efficient Operation of the Buggies and Affords Ample Protection Against Blowing and Spilling

E. Two Men Dumping Cement Buggies Simultaneously. On This Job 2-Batch Trucks Received Their Supply of Bulk Cement in from 7 to 7½ Seconds

F.—A Bulk Cement Batching Plant. On This Job the Cement Was Delivered by Trucks Direct from the Mill

for each batch. Figures 5, A and 5, B show the layout for two yards using two bins and bulk cement. Figure 5, C shows an extremely simple layout for a batching plant using one size coarse aggregate and bag cement. Table 3 gives the yard time constants for these three layouts.

Storage of bag cement sufficient for at least one day's run should always be provided unless prompt car arrivals can be guaranteed. For a fast job this will ordinarily require a shed or platform having a floor area of about 550 square feet. Usually the storage is in the form of a shed forming a part of the regular loading platform so that the bags can readily be shifted by hand trucks from the car to the shed for storage and again, when necessary, from the shed to the batch trucks.

Equipment for the mechanical handling of bulk cement is used by many contractors. It would seem desirable that the size of the bin used with these plants be enlarged so as to provide sufficient capacity for one full day's run of cement and thus do away with the necessity of using bag cement as emergency storage. The bin should also be erected so as to permit a drive straight through and thus help to keep down the time constant or loading time of the batch trucks. The following tabulation gives the average operating characteristics of a typical cement-loading plant.

	Seconds
Fill cement hopper	17.2
Dump cement into truck	10.5
Average cycle	27.7

TABLE 3.—*Yard time constants, in seconds, for layouts shown in Figure 5*

Item	Layout A		Layout B		Layout C: 2-batch trucks
	1-batch trucks	2-batch trucks	1-batch trucks	2-batch trucks	
Loading aggregates	10	68	11	70	27
Loading bulk cement and adjusting tarpaulins	33	40	33	53	
Loading bag cement					37
Driving within yard	90	110	51	59	71
Turning and backing	74	97	22	30	
Fixing batch boards		18		19	
Total net yard constant	207	333	117	231	135
Average waits and delays					15
Total gross yard constant					150
Driving distance within yard, in feet	800	800	875	875	250

The hopper could always be filled while the trucks were being exchanged so that the average time the 3-batch trucks required for taking on the cement was 73 seconds.

The loading yard must provide for the shortest and most direct possible routing of the trucks through the yard so as to detain each one as little as possible; but, above all, the yard and loading plant must provide for the maintenance of a regular uninterrupted service. Each minute added to the time the truck must spend in the yard means a cost of from 2 to about 5 cents, depending on the size of the truck and the rate paid for drivers. This charge occurs each time a truck passes through the yard. Thus, if 2-batch trucks supplying 400 batches per day to the mixer are each worth 4 cents a minute, and if the yard layout is such that to serve each truck requires an average of 2 minutes more than another layout would have required, then the loss is $16 per working day so long as this yard is used.

Costly as are the faults which increase the time required for serving the hauling units, those faults or conditions of yard layout, arrangement, or equipment which contribute to the irregularity of their operation are more to be dreaded. Any irregular delay to the trucks at the loading plant is also likely to be imposed on the mixer, and mixer delays are far more expensive. While a regular loss of truck time costs anywhere from 2 to 5 cents per minute, delays to the mixer entail a cost of about 75 cents or even $1 per minute.

Such losses are not to be looked upon lightly. In fact, an occasional delay averaging only one minute each hour to the mixer is likely to be as expensive as a regularly occurring delay of one minute to each truck loaded out of the yard. The first and most essential object of the material plant must therefore be to facilitate regular, continuous operation of the hauling units well within the time intervals set by the mixer.

The yard layout should therefore be such that, first of all, regular, continuous service can be readily maintained; second, that each hauling unit will be detained in the yard the shortest possible time interval in obtaining its load on each trip; and, third, that no regular single stop in the yard will exceed a period equal to the length of the mixing cycle multiplied by the number of batches carried by the truck. Otherwise, the yard instead of the mixer becomes the pacemaker.

Care should be taken to arrange the routing of the trucks through the yard so that there will be no interference between the various units. Since continuity and regularity of service are the prime essentials, considerable thought should be given to the location and maintenance of the road or driveway through the yard. The development of mudholes in which trucks might get stuck or of chuck holes endangering springs or parts, can not be tolerated, for a stuck or disabled truck generally means a delay to the mixer representing a loss of from 75 cents to a dollar per minute, despite the fact that the delay to the truck itself means the loss of only a few cents per minute. Thus, a long or slow passage through the material yard, so long as there is nothing to prevent orderly and regular progress, is not so serious. Even the worst yard condition of this kind would probably not mean more than one additional large truck or about $25 per day. On the other hand, three 10-minute mixer delays during the course of the day will probably exceed this loss.

As examples of the actual effect yard layout and management have in determining the length of time required for each vehicle to take on its load and pass through the yard, 24 rather typical jobs were selected, one half having a fairly good yard layout and able management and the other half with poor yard layouts and only fair to poor management. In this selection jobs using 1, 2, 3, and 4 batch trucks were all represented by 6 jobs for each size, one-half of which were good, well-managed layouts and the others inferior, poorly-managed layouts. These are given in Table 4, together with a comparison, for the same items, of the average values from over a hundred paving jobs having all kinds of layouts and grades of management.

It will be noted that the time differential in favor of the good as against the poor layout averages about 4 minutes, while the good layouts also show a differential of from about 1 to nearly 3 minutes below that of the average yard constant for all jobs. These are items too large to be ignored, as they may readily mean an unnecessary but permanent addition to the hauling equipment of at least the equivalent of one 3-batch truck.

TABLE 4.—*Effect of yard layout and management on magnitude of time constant*

GOOD YARD LAYOUT AND ABLE MANAGEMENT

Number of studies averaged	3	3	3	3
Size of trucks	1-batch.	2-batch.	3-batch.	4-batch.
Load aggregate ... seconds	10	38	70	98
Load cement bags ... do	12	29	54	84
Drive and maneuver ... do	48	65	87	62
Total net yard constant ... do	70	132	211	244

AVERAGE YARD LAYOUT AND MANAGEMENT[1]

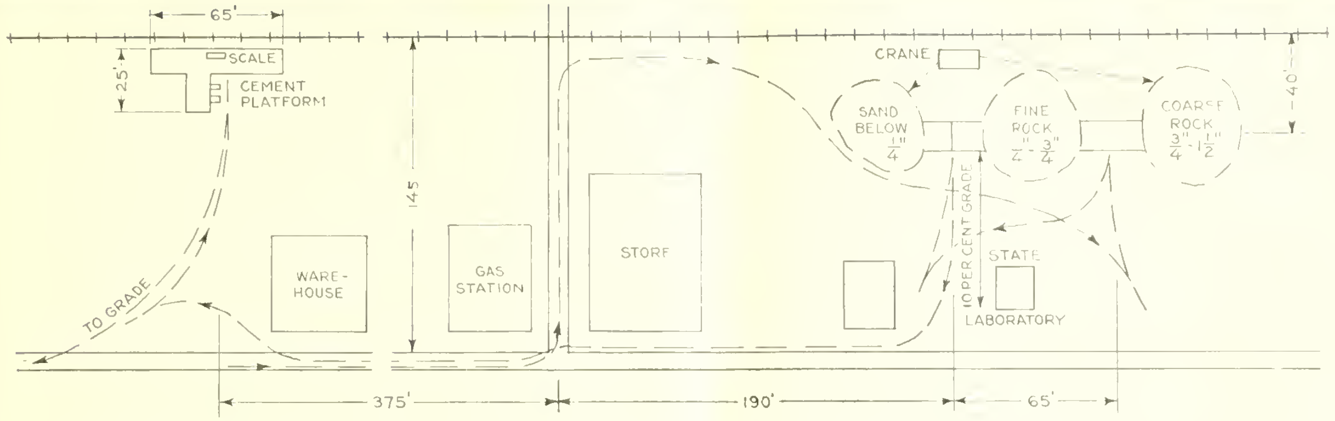
Size of trucks	1-batch.	2-batch.	3-batch.	4-batch.
Load aggregate ... seconds	15	59	114	190
Load cement bags ... do	28	80	106	110
Drive and maneuver ... do	81	99	102	100
Total net yard constant ... do	124	238	322	400

POOR YARD LAYOUT AND SLACK MANAGEMENT

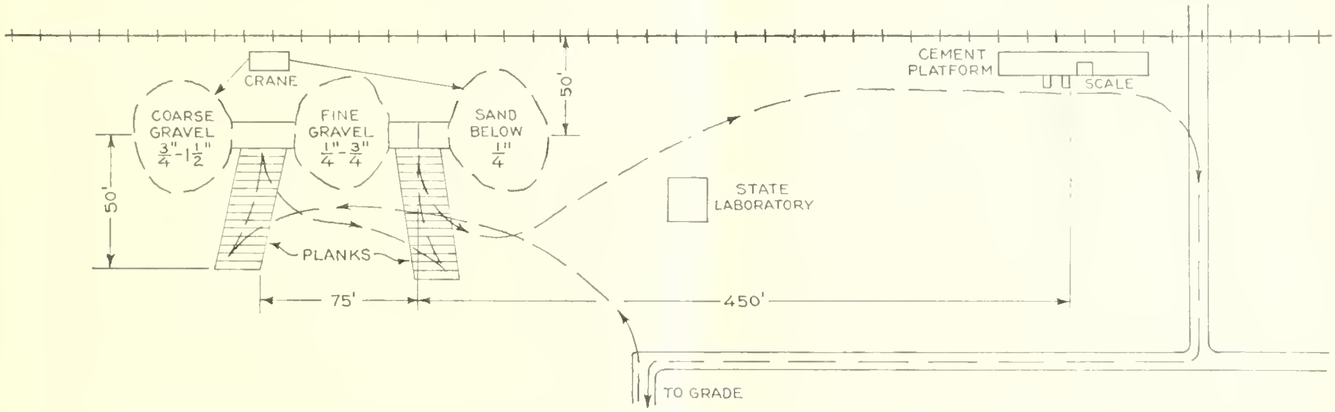
Number of studies averaged	3	3	3	3
Size of trucks	1-batch.	2-batch.	3-batch.	4-batch.
Load aggregate ... seconds	33	91	163	196
Load cement bags ... do	51	89	118	148
Drive and maneuver ... do	210	218	193	132
Total net yard constant ... do	294	398	474	476

[1] Average values for more than 100 jobs.

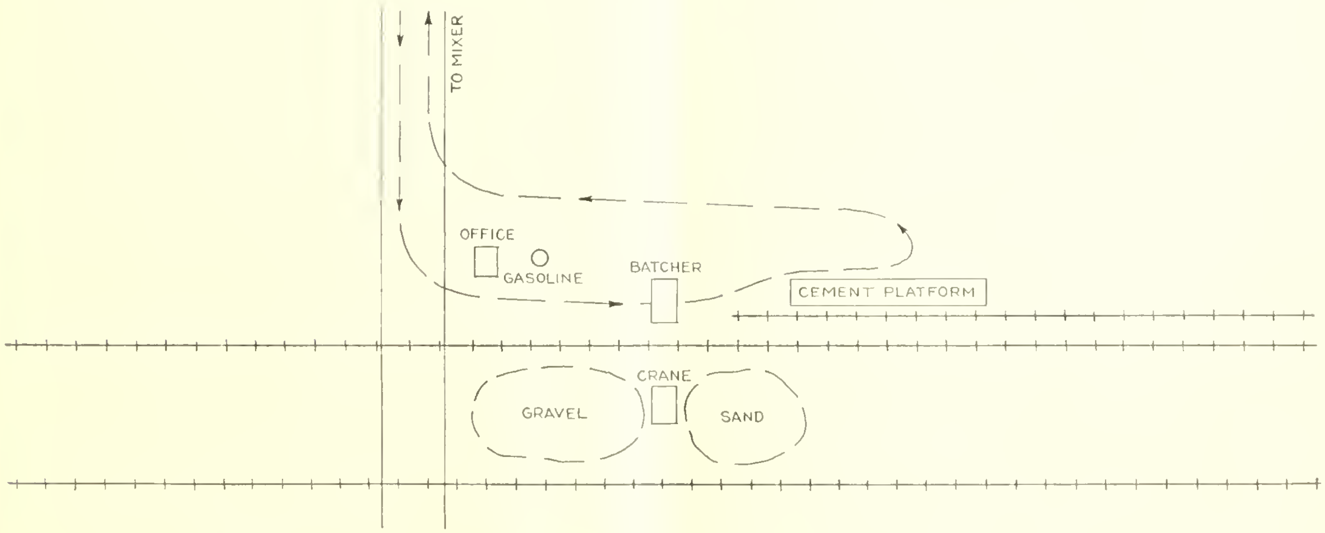
Every maneuver or operation within the yard requires time. To complete the usual 90° turn and back the truck under the bins ordinarily consumes from 20 to 30 seconds. With good batching facilities the actual standing time for a truck in taking on one batch of sand and coarse aggregate may regularly be as low as 10



A. TIME-CONSUMING YARD LAYOUT; TWO SIZES OF COARSE AGGREGATE; BULK CEMENT HANDLED WITH BUGGIES.



B. IMPROVED YARD LAYOUT; TWO SIZES OF COARSE AGGREGATE; BULK CEMENT HANDLED WITH BUGGIES



C. CONVENIENT YARD LAYOUT FOR ONE SIZE OF COARSE AGGREGATE AND BAG CEMENT

FIGURE 5.—EXAMPLES OF YARD LAYOUTS

seconds and need rarely exceed 20 seconds. The time required for multiple-batch trucks is much longer, so that from 20 to 40 seconds will usually have to be added for each additional batch. The use of two bins instead of one for loading the sand and aggregates is therefore found to increase the yard time constant of the trucks about 40 to 50 seconds for the 1-batch trucks, 75 to 90 seconds for 2-batch trucks, and from about 100 to 125 seconds for the 3-batch trucks under fair to good management and operating conditions when it is necessary to back under the bin. If the drive is straight through these figures can generally be reduced by from 20 to 40 seconds. This extra time for taking on the subsequent batches is due to the fact that the first batch can

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ordinarily be weighed out in the hopper and made ready to dump while the truck is coming in. But after this first batch is dumped the truck must then wait while the following batch is weighed out, which ordinarily requires from 20 to 30 seconds provided the sand flows readily.

Driving speed within the yard is, of course, low, seldom more than an average of 250 to 400 feet per minute. The average yard seems to have a yard loop or circuit which requires about 1½ minutes for the actual driving. The variation of individual yards from this average, however, is very large. The length of the yard circuit varies greatly while the driving conditions vary even more. Yards have thus been found which had a driving time as low as 35 seconds and others as high as 5 minutes. The importance of reducing the length of the driving circuit and improving the condition of the road should be apparent.

In a large contract there is often a possible choice of two or more locations for the loading plant. The question, then, arises as to whether to build entirely from one plant set-up or move to another as the work progresses. The cost of hauling of course varies as the length of haul plus the loading and unloading costs. But, to the concrete-paving contractor, the factor of regularity in the operation of the hauling units is of utmost importance. Thus, short hauls employing less vehicles are usually easier to organize into a smooth-functioning unit. While some surplus must always be maintained as insurance, the chance that this surplus will prove inadequate is always greater on the long hauls.

To ascertain the proper plant location and whether more than one should be used requires a knowledge of both hauling costs and the cost of dismantling, moving, and again assembling the plant at the new location. The cost of the hauling can be forecast with considerable accuracy provided the rental rates or rental value, the operating characteristics of the trucks which are to do the hauling, and the conditions under which the hauling is to be done are known. Of these, the operating characteristics are the most important as well as generally the most diffcult to forecast, since they depend largely on both the yard layout and road or hauling conditions as well as the efficiency of the management. However, for any given job these factors can all be determined within reasonable limits by means of actual observations.

Having these data the following rather simple formula is suggested for forecasting the probable cost of hauling the batches for all or any desired part of the contract. If the cross section of the slab is not constant each portion so varied may be taken as a separate unit and the total cost will then be the sum of the several sections. The formula is as follows:

$$K=\frac{C}{60N}\left[T+\frac{60}{S}\,(L_1+L_2)\right]W \qquad (1)$$

Where K = the cost of the hauling for section (L_1-L_2).

C = the hourly cost or rental value for the size and type of truck to be used.

N = the number of batches carried per load, or if tons per load W is also given in tons.

T = the total truck constant or average time the truck spends each trip at the mixer and in the yard, including all necessary and regular delays, in minutes.

S = the average actual round-trip speed in miles per hour while the truck is on the road.

L_1 = the dead-haul or hauling distance in miles from batcher plant to beginning of concrete slab being laid.

L_2 = the total length of haul in miles from batcher plant to end of the section under consideration.

W = the total number of batches (or tons if N is given in tons per load) to be hauled for the section under consideration. If the cross section is not uniform a separate computation should be made for each distance involving a changed section length.

With a slight modification this formula can also be used for forecasting the cost of hauling per batch-mile or per ton-mile, or the total cost of hauling a batch or a ton any given distance, L. The cost of hauling per batch-mile or ton-mile is given by the expression,

$$B=\frac{C}{60N}\left(\frac{T}{L}+\frac{120}{S}\right) \qquad (2)$$

and the total cost of hauling a batch the distance L is given by the expression,

$$D=\frac{C}{60N}\left(T+\frac{120}{S}\,L\right) \qquad (3)$$

in which B = the cost of hauling per batch-mile when N is batches per load or per ton-mile when N is tons per load.

D = the total cost of hauling a batch or ton the distance L.

L = the actual length of haul, including dead haul.

All other factors are the same as in formula 1.

In determining whether or not the hauling should actually be done from two or more yards instead of from one, the following points should also be given careful consideration in addition to the solution of the cost of the haul itself:

1. The cost of moving and setting up on the new location.
2. The cost of materials laid down at the new yard.
3. Track and switching facilities at the new yard.
4. Adaptability of the available space for a yard layout.
5. Hauling and traffic conditions from the proposed location.
6. Probable time constant for new yard and yard operation cost.

Each of these items may have a very definite bearing on whether the move should or should not be made; yet, each case is likely to have so many special conditions that general rules are apt to prove of little value. Thus the actual total cost of making the move and getting ready to operate from a second location only 5 or 10 miles distant may readily vary from $500 to $5,000, while the actual time the paving work will be down may vary from one day to more than a week. No simple rule, such as the stipulation that if the haul can be reduced by some given amount a second yard should be used, will suffice to meet the varying conditions. Every case must be carefully investigated. Such items as freight rates, land rentals, switching and track facilities, as well as reliablity of delivery of materials are seldom the same for any two possible locations and may be sufficient to overbalance any possible saving from the shorter haul. The second location, for example, may be on a different railroad, so that the freight rates may be either higher or lower by a considerable amount;

or, again, the rates may be the same, but the new location may involve the transfer of the cars in transit from one system to another at some point where exchanges are more or less infrequent and therefore likely to make delivery slow and irregular. Such a location would be undesirable unless space were available for more than normal stock piling.

Sometimes siding facilities are inadequate, especially when two or more sizes of coarse aggregate are required. The installation of an additional spur is usually rather expensive. The work involved in providing a satisfactory driveway, as well as suitable locations for the bins, cement platform and storage shed, and stock-piles is seldom the same in any two cases. Careful advance investigation will nearly always produce the data from which the cost and time required for developing the new location can be estimated with reasonable accuracy.

Another item which must not be overlooked is that of traffic conditions in connection with the new yard. Congested highway traffic is always bad in that it limits speed and tends toward irregularity. Railroad grade crossings over main lines carrying heavy traffic are always dangerous, while those over freight-passing sidings are apt to occasion frequent and sometimes long delays. Conditions have been found where as many as two extra 3-batch trucks and a crossing watchman were required to prevent serious delays to the mixer. This is equivalent to a tax of 12 to 15 cents on every batch handled from such a yard when production is high, and correspondingly more when production is low.

In general, we find that for any influence which increases the truck time constant the capacity of the hauling equipment must be increased one batch for each mixing cycle added to the length of the time constant. Thus, if the regular mixing cycle is 1¼ minutes, then an increase of 2½ minutes in the time constant on a job using 2-batch trucks will require the full time addition to the hauling equipment of one additional 2-batch truck of the same type and speed as those used on the job.

The actual process of determining whether a second yard should or should not be used for any particular job might be illustrated as follows: Contractor A has a job 7 miles in length which will require 2,000 batches per mile. One end of the job is a half-mile from the siding where the materials plant can be located with ample room for storage. A second siding where an equally advantageous yard can be located and where hauling conditions are similar to the first, is 2½ miles beyond the far end of the road. The question before the contractor is whether or not the saving in hauling costs from two set-ups will exceed the cost of making the move. The contractor, after examining the proposed location and securing data as to freight rates, ground rentals, etc., makes his estimates about as follows.

He is using 3-batch trucks which are costing him at the rate of $3 per hour. For this location these trucks will have a time constant of about 10.5 minutes and maintain an average round-trip speed of 20 miles per hour, which is the same as at the present yard. Freight rates and switching charges happen to be the same for both sidings. The cost of moving the yard and making the new set-up will require two working days. His straight-time pay roll amounts to $65 a day, and about $60 a day for additional labor will be required during the two days of moving and setting up at the second yard, making the direct labor cost $250. After allowing for salvage value of lumber at the old yard, the cost of materials and erection of the cement loading platform, shop, office, and storage shed will amount to $250. Loss of material in cleaning up base of stock piles, he estimates at $100. Incidental materials, gas, oil, etc., are placed at $100. Ground rental and miscellaneous items add another $100. The cost of moving the yard will therefore be $800, aside from the loss of two working days and a temporary disorganization of his working force.

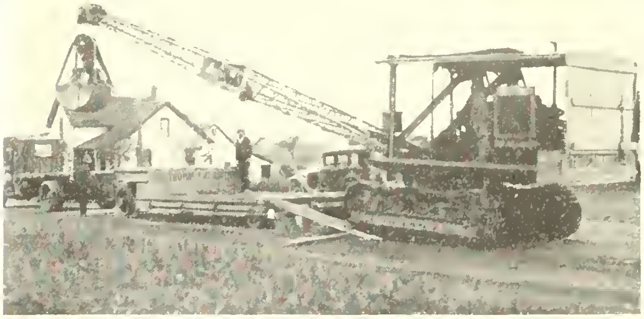


Figure 6.—The Use of Heavy Trailers Sometimes Facilitates the Moving of Heavy Equipment

Against this cost he must balance the possible saving to be secured from a reduced hauling distance for 2½ miles of the contract, so that his maximum haul, instead of 7½ miles from the one set-up, will be only 5 miles from each of the two set-ups. Substituting the above data in formula No. 1, we find that the probable cost for hauling the entire job from the first yard would be $8,050, while the cost of hauling for 4½ miles from the first and 2½ miles of the job from the second yard would be $6,800, a margin of $1,250 in favor of using the two yards. Since he finds that the cost of making this move would probable not exceed $800, the move in this case is justified.

The many and variable factors which enter into batcher plant operation make the drawing of general conclusions very difficult and generally impossible. An analysis of the data secured by means of detailed stopwatch studies on more than a hundred going jobs all serve to direct attention to this one feature: That the attainment of low unit costs in handling, batching, and hauling the materials which enter into our concrete roads is not a matter of chance or of luck, but the result of careful advance planning and constant supervision by an able and alert management.

INDEX TO VOLUME 12 OF PUBLIC ROADS AVAILABLE

An index to volume 12 of Public Roads, which includes the issues from March, 1931, to February, 1932, is now available for distribution, and copies may be obtained from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C. Indexes to volumes 6, 7, 8, 9, 10, and 11 have previously been published, and a supply of these indexes is still on hand.

CONNECTICUT AVENUE EXPERIMENTAL ROAD NOW 20 YEARS OLD

THE CONNECTICUT Avenue experimental road, extending from Chevy Chase Circle to Chevy Chase Lake in Montgomery County, Md., was built during the years 1911, 1912, and 1913 by the Bureau of Public Roads. The road was constructed as two separate projects, the dividing line being at Bradley Lane. South of that point sections were constructed of water-bound macadam with subsequent bituminous surface treatment and also sections of bituminous macadam. North of Bradley Lane the sections consisted of Portland cement concrete and of bituminous concrete and brick on Portland cement concrete foundation.

All of these experimental sections are now either 19, 20, or 21 years old and all are in service under a heavy traffic. Beginning with the present issue a series of photographs is to be published on the cover of Public Roads showing sections of the various types of construction immediately after completion and their appearance to-day after the lapse of 20 years.

The section chosen for this issue is a portion of experiment 3, which is composed of penetration bituminous macadam in which a fluxed native asphalt was used. This project was constructed in 1911. The photographs on the back cover include a construction picture and a view of the finished pavement several months after completion. The front cover shows its present condition.

The initial cost of this section of bituminous macadam was 64.69 cents per square yard. In 1918 a surface treatment costing 20.84 cents per square yard was applied. Maintenance costs prior to 1918 totaled 1.44 cents per square yard. Since 1918 a total of 9.62 cents per square yard has been spent for maintenance. The traffic has been increasing steadily since the road was built, and in 1931 reached an average density of over 4,000 vehicles per day.

It is recognized that there are many other roads in the country which have reached an age of 20 years or more. The bureau would be glad to receive from the readers of Public Roads descriptions, with photographs, of road surfaces which have stood the test of time.

(Continued from p. 96)

Figure 1. This shows how dissimilar asphalts and distillates may be combined to produce finished cut-back products of identical character and demonstrates why the consumer should not attempt to control the characteristics of the original components of a cut-back or blended product.

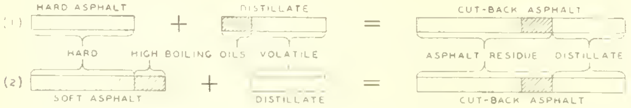


Figure 1.—Diagram Showing How Cut-Backs of Identical Character May Be Formed From Dissimilar Asphalts and Distillates

CONCLUSION

The liquid asphaltic road materials which are involved in this cooperative undertaking are manufactured and sold by a great number of producing companies which, from the commercial standpoint, may not always have a common interest. They are furnished in vast quantities, in a number of grades and for varied purposes, to practically every State and are required to meet specifications which, to a great extent, are peculiar to the individual State and have been developed by the State as a result of its own experience. When one considers the conflicting interests and the varied opinions which exist; the general acceptance of the many tests which have been in long-continued and unquestioned use; and the somewhat violent challenge to current practice which is contained in the proposed simplified scheme of analysis, the degree of approval which has been given it after only a year of study, and the degree to which tests outside its scope have been eliminated, may be considered to be truly remarkable. This accomplishment would not have been possible without the wholehearted cooperation which has been given by the manufacturers and the State highway departments. That all have contributed so generously to the work is evidence of the keen appreciation that drastic changes in present practice are to be desired.

Much remains to be accomplished before the desired degree of simplification and standardization of a common scheme of analysis can be effected, and beyond this there is the further need of agreement regarding specification test limits which will establish a series of standard grades of material for the various types of construction. The progress which has been made gives promise of ultimate success in securing the needed improvements in existing practice.

CORRECTIONS

Vol. 13, No. 2, April, 1932.—In the article entitled "The Resistance of Concrete to Frost Action," page 35, it was erroneously stated that aggregate No. 5 was a gravel from Millville, N. J. This gravel was obtained from Farmingdale, Monmouth County, N. J.

Vol. 13, No. 3, May, 1932.—In the article entitled "Concrete Pavement Design Features, 1931," the table on page 50 gives the width of concrete pavement on projects submitted by the State of Washington as 18 feet. The standard width in Washington is 20 feet. Out of 67 miles of concrete paving on Federal-aid projects submitted by this State during 1931, there were 63½ miles of 20-foot pavement and 3½ miles of 18-foot pavement.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

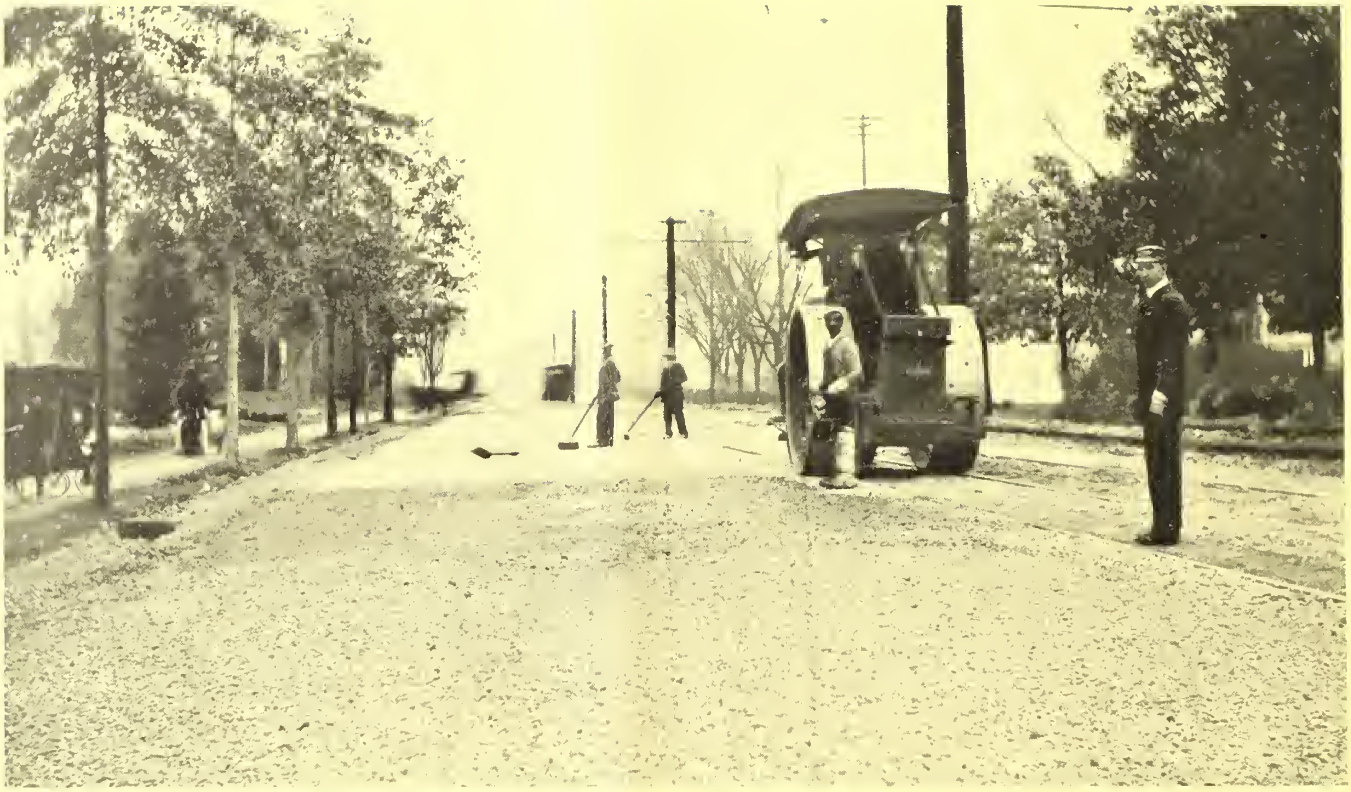
CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

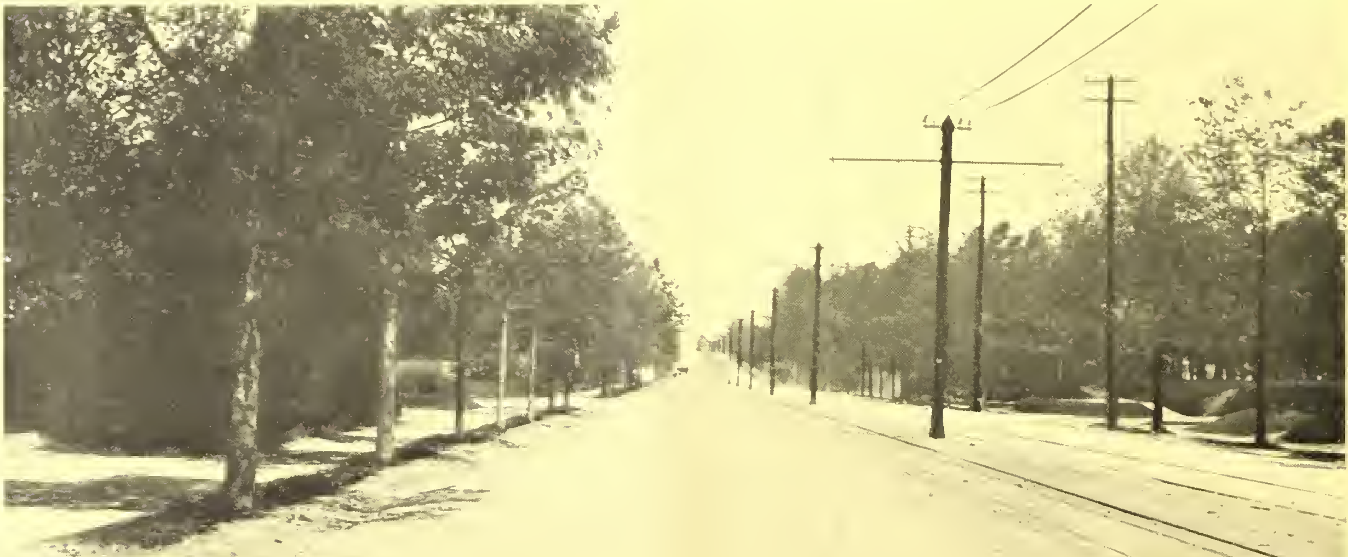
JULY 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
					Initial	Stage¹	Total			Initial	Stage¹	Total		
		$	$					$	$				$	
Alabama	2.405.2	1,638,442.00	797.324.95	96	62.8		62.8						5.469.694.24	Alabama
Arizona	1,229.4	2,431.284.00	1.750.127.23	70	94.4	61.7	156.1	4.594.57	3.459.24		0.3	0.3	810,004.99	Arizona
Arkansas	1.960.2	2.451,768.24	1.127.328.92	86	64.8	45.1	109.9	362.032.53	181,016.26	17.3		17.3	2,183.603.25	Arkansas
California	2.359.9	9,507.941.91	4.533.068.35	55	209.3	49.5	258.8	610.039.47	188.171.69	27.5	4.4	31.9	130,615.78	California
Colorado	1.632.1	3,754.986.86	1.987.694.54	78	155.0	45.7	200.7						2,058,274.18	Colorado
Connecticut	287.0	4.071,872.74	1,479,495.06	76	24.9		24.9	710.239.12	337.234.32	10.3		10.3	329.792.14	Connecticut
Delaware	363.2	711,647.00	355.823.50	70	23.7		23.7	86.524.00	43.262.00	4.7		4.7	167,140.82	Delaware
Florida	641.4	3,489.185.36	1,613.648.03	86	99.1		99.1	658,535.24	322.435.77	17.3		17.3	2.544,886.16	Florida
Georgia	3.081.0	6.400,105.13	2,994.218.88	69	149.3	167.0	316.3	127.270.50	63.635.24	4.6	1.2	5.8	1.351.969.22	Georgia
Idaho	1.417.6	2,524.336.26	1,495,426.51	76	112.7	149.6	262.3	1,283.119.89	770.043.65	45.1	184.0	229.1	156,658.16	Idaho
Illinois	2.702.4	25.150.518.35	11.663.716.64	79	757.6	36.8	794.4	2.354.574.70	1,003.165.26	79.9	5.3	85.2	1.332.521.57	Illinois
Indiana	1,851.0	10,346.805.27	5.130,201.91	45	378.7	20.5	399.2	761.496.48	380.730.95	36.0		36.0	341.205.50	Indiana
Iowa	3.324.8	5.412.086.54	2,525.295.18	37	255.1	65.7	320.8	714,471.56	231.710.02	30.2	18.9	49.1	42.800.00	Iowa
Kansas	3.620.5	5.277,872.95	2.593.581.92	67	296.3	99.9	396.2	1,834,697.90	887,701.29	61.6	267.8	329.4	560,318.87	Kansas
Kentucky	1.892.3	2.645.916.04	1,187,938.37	74	161.6	16.0	177.6	1.627,756.73	633.752.98	38.8	91.2	130.0	983,390.78	Kentucky
Louisiana	1.567.5	7,456,322.25	3.503.393.25	49	87.0	9.9	96.9						868.290.73	Louisiana
Maine	721.3	3.981,360.08	1.735,005.26	65	116.3		116.3	104,593.33	52,296.66	.3		.3	191,004.42	Maine
Maryland	778.9	1.170.603.98	484,776.57	58	38.9	.6	39.5	1,149.155.55	462.712.12	45.0	5.6	50.6	8,817.81	Maryland
Massachusetts	817.3	7,129,265.67	2.897.641.92	80	71.3	.2	71.5	285.726.10	142.863.04	2.2	6.5	8.7	459.748.73	Massachusetts
Michigan	2.084.4	9.978.642.29	4,578.493.99	76	391.6	31.9	423.5	1,162.226.40	545.502.50	21.2	45.5	66.7	1.772,525.53	Michigan
Minnesota	3.948.2	11.543,711.88	3.804,301.38	58	205.0	304.7	509.7	583.201.84	175.000.00	15.6	12.3	27.9		Minnesota
Mississippi	1.809.0	3,875.720.84	1,898.233.36	87	171.2	68.8	240.0	75.492.84	37.746.42	16.1		16.1	5.191.098.68	Mississippi
Missouri	2.928.7	7,007,407.19	3.040.641.57	51	185.0	97.1	282.1	361.486.67	173.742.54	18.7	2.9	21.6	562.565.24	Missouri
Montana	2.759.5	3.972.279.91	2.230.226.09	79	362.5	31.5	394.0	275,936.51	154.007.78	5.7	98.4	104.1	3.299,347.07	Montana
Nebraska	4.284.9	2.794.323.43	1,386,098.91	71	122.1	26.1	148.2	1.813.356.01	841.768.57	28.2	111.6	139.8	1.861,142.54	Nebraska
Nevada	1.256.6	2.218.423.33	1.887.476.60	63	40.6	187.6	228.2	194,252.66	172.521.31	7.4	39.9	47.3	10.830.62	Nevada
New Hampshire	421.0	548,058.81	244.966.84	54	17.1	2.5	19.6	41,149.53	20,574.76	2.4		2.4	428,596.59	New Hampshire
New Jersey	623.6	4,314.673.59	1,554,698.78	53	27.9	.5	28.4	1,131,894.01	565,947.00	.3		.3	603.796.98	New Jersey
New Mexico	2,239.0	1.692.957.16	1.189.247.98	79	78.3	31.4	109.7	690,644.66	409,067.23	22.2	29.1	51.3	906.657.37	New Mexico
New York	3.280.6	16.525,600.00	7,429,600.00	50	402.8		402.8	4.894,980.00	1.824.190.00	106.6		106.6	1,659,610.27	New York
North Carolina	2.226.4	1.199,041.83	590,635.84	77	68.0	5.0	73.0	310,669.92	154,517.62	6.9	20.9	27.8	3.701,148.36	North Carolina
North Dakota	5.016.0	3.759.681.72	1,885.597.27	53	483.5	338.0	821.5	1.479.246.33	747.558.95	104.4	427.8	532.2	762.530.67	North Dakota
Ohio	2.879.7	6.861.611.07	2,236.780.15	68	132.4	25.6	158.0	6.161.923.90	2.336,854.89	156.0	58.4	214.4	929.626.00	Ohio
Oklahoma	2.303.6	3.668,796.96	1.727.629.03	78	184.3	36.7	221.0	2.100,324.42	1.081.218.89	112.5	17.6	130.1	870.605.64	Oklahoma
Oregon	1,558.4	3.597.543.02	1,989.194.21	75	99.0	43.1	142.1	1.030.058.22	586.479.02	44.3	65.6	109.9	899.554.97	Oregon
Pennsylvania	3,025.5	6,916.867.26	3.295.528.18	29	277.1		277.1	4.390.531.74	1.924.475.86	116.5		116.5	511,078.08	Pennsylvania
Rhode Island	257.8	802,583.73	397.179.71	73	16.6		16.6						326.325.34	Rhode Island
South Carolina	2,049.9	2.366.835.31	1.065.169.32	98	60.4	41.4	101.8						744.636.09	South Carolina
South Dakota	4.004.6	3.846.371.88	2,262,747.75	65	304.9	218.6	523.5	198.838.04	110.414.74	13.2	27.1	40.3	799.610.50	South Dakota
Tennessee	1.670.5	1.804.913.59	885,932.91	54	61.9	19.4	81.3	559,531.22	279.080.49	10.2	8.7	18.9	3,012.060.90	Tennessee
Texas	7.757.2	14,518.790.54	6.661.286.22	71	657.2	261.8	919.0	4.606,578.85	2,104,664.82	240.0	220.0	460.0	2,513.758.06	Texas
Utah	1.164.4	1.341.973.90	974,822.93	67	90.0	66.8	156.8	656.678.26	496.284.08	49.2	69.2	118.4	482.931.79	Utah
Vermont	339.1	1,416.419.92	634.311.33	35	45.7		45.7							Vermont
Virginia	1.922.4	1,914.628.55	957.314.21	75	112.3	6.2	118.5	814,945.19	371.890.68	30.3	21.6	51.9	1.766.774.90	Virginia
Washington	1.216.8	2.846.109.67	1.355,994.35	70	92.0	14.5	106.5	837.651.47	305,300.00	17.9	5.3	23.2	824,469.51	Washington
West Virginia	893.3	1.893.304.80	823.391.31	77	48.7	10.5	59.2	952,257.00	466.971.47	33.4		33.4	609,263.59	West Virginia
Wisconsin	2.598.5	7.774.738.25	3,190.598.32	55	206.7	70.7	277.4	1,003.163.91	393.400.00	29.5	20.1	49.6	335.738.52	Wisconsin
Wyoming	1,942.3	3.425.976.48	2.135.753.94	75	176.4	225.1	401.5						123.028.57	Wyoming
Hawaii	76.3	876.665.56	460,511.05	61	23.5		23.5	674,545.52	533.085.24	17.2	3.6	20.8	1.335,908.87	Hawaii
TOTALS	101.191.2	240,856.973.10	112,630,070.52	65	8.303.5	2,933.7	11,237.2	49.676.392.79	22.516.455.35	1,646.7	1.890.8	3.537.5	56,835.958.60	TOTALS
Construction completed		155.701,000	73,383.000											
Balance uncompleted		85.156.000	39.247.000											

¹ The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.



Connecticut Avenue Experimental Road, Chevy Chase, Md., November, 1911, Showing Application of Seal Coat in the Foreground and Finished Bituminous Macadam Road in the Distance



Surface Condition of Bituminous Macadam Section in May, 1912. The Photograph on the Front Cover was Taken at the Same Location

U. S. GOVERNMENT PRINTING OFFICE 1932

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 7 SEPTEMBER, 1932

A BITUMINOUS CONCRETE ROAD ON ITS TWENTIETH BIRTHDAY

For sale by the Superintendent of Documents, Washington, D. C. See page 2 of cover for prices

PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 7

September, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

STATIC LOAD AND IMPACT TESTS OF LIGHTWEIGHT BRIDGE FLOOR SLABS

REPORT OF A COOPERATIVE INVESTIGATION CONDUCTED BY ALLEGHENY COUNTY, PA., AND THE UNITED STATES BUREAU OF PUBLIC ROADS

By L. W. TELLER, Senior Engineer of Tests, and G. W. DAVIS, Associate Engineer of Tests, United States Bureau of Public Roads

THE Division of Tests of the Bureau of Public Roads has recently completed a series of static load and impact tests on two bridge floor slabs of a rather unusual type. These tests were undertaken at the request of the Bureau of Public Works of Allegheny County, Pa., and were carried out as a cooperative project with that organization.

The slabs tested are a combination of steel shapes in the form of a grating or mat which, after erection, is filled with Portland cement concrete. The features of particular interest are the small depth, 3 inches; and the moderate weight, between 50 and 55 pounds per square foot. The advantages of a bridge floor slab having these characteristics are obvious.

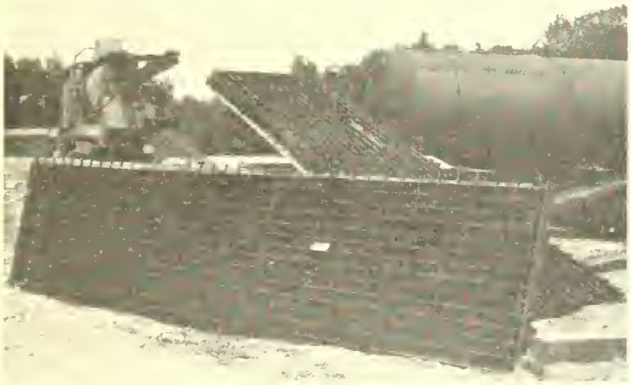


FIGURE 1.—APPEARANCE OF FACTORY-ASSEMBLED MATS FROM WHICH FIRST SLAB WAS CONSTRUCTED

Static load tests on small sections of a slab of this general type had indicated that such construction possesses flexural strength sufficient to warrant its consideration for use in the design of highway-bridge floors,[1] but there was some question as to whether or not the impact of the wheels of motor vehicles might break the bond between the steel and the concrete, thus weakening the slab structurally and also permitting the entrance of moisture between the two materials.

The primary object of the tests carried out by the Bureau of Public Roads was to determine to what extent severe motor-vehicle impact would affect the structural strength of bridge floor slabs of this type.

To obtain this information a definite schedule of static loads was applied to the slabs before, during, and after the program of impact loadings to which they were subjected. Deflection and strain measurements for each static loading made possible comparisons of the structural action of the slabs before and after receiving impact.[2]

DETAILED DESCRIPTION OF THE SLABS

While the two slabs tested were of the same general type and appearance, they differed from each other in a number of details and are therefore described separately. The slab built and tested in 1930 is referred to as the first slab and that built and tested in 1931 as the second slab.

The first slab was 12 feet, 7½ inches by 14 feet, 10½ inches in area and 3 inches thick. It was made up of five factory-assembled mats 35¾ inches wide and 12 feet, 7½ inches long. (See Fig. 1). Each of these mats was composed of 13 T-bars each with a 2½-inch base or flange and a 3-inch stem. These were assembled with their bases in contact, side by side, forming the bottom of the mat. The stems were locked together by means of 1 by ⅛ inch flat cross bars which were pressed into curved slots cut in the vertical stems of the T-bars. These cross bars were placed on 4-inch centers. Figure 2 shows the details of this construction. It will be noted that the slots in alternate T-bars curve in opposite directions. The cross bars are forced into these slots under heavy pressure and are caused to twist in opposite directions at the successive junctions with the T-bars, securely locking the whole upper surface in place. As these slots are above the neutral axis of the T-bars, the lower section, which is subject to tension, is left intact and the compressive strength of the upper section is maintained presumably by the wedging action of the cross bars. In those sections of the mats above the supporting stringers, where negative moments are encountered, the position of four cross bars was changed, the slots being punched through the stems halfway between the upper and lower surface of the mat, as shown in Figure 2.

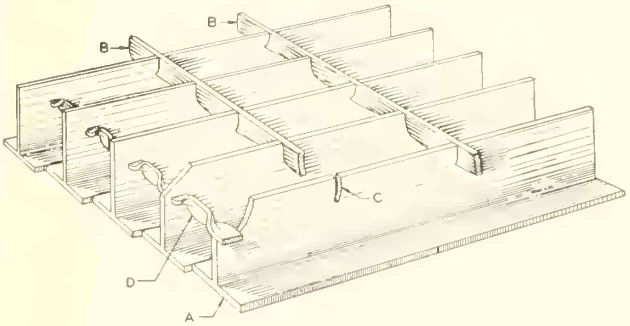

B
B
C
D
A
A – BASE OF T-BARS
B – CROSS BARS BETWEEN STRINGERS
C – CURVED SLOTS
D – CROSS BARS ABOVE STRINGERS

FIGURE 2.—ASSEMBLY OF STEEL SECTION OF FIRST SLAB

The steel mats were placed transversely on the stringers and were attached to them by spot welding through notches cut in the edge of the base of every other T-bar, to the center of the flange of the stringer.

The abutting edges of adjacent mats were bolted together at points midway between the stringers by five ½-inch bolts spaced 4 inches on center and passing through the stems of the T-bars which formed the edges of the mats. The result of this construction was a smooth grated surface with openings or pockets 2½ by 4 inches in size, the appearance of which, before being filled with concrete, is shown in Figure 3.

The concrete used in the first slab was of 1:1.5:3 proportions (by dry-rodded volumes). The water-cement ratio was 0.86. The aggregates used were Potomac River sand and a siliceous gravel from Fredericksburg,

---

[1] Engineering News-Record, vol. 104, No. 2, Jan. 9, 1930. Development tests on a light floor for bridges, by Leon S. Moisseiff.

[2] For a description of another investigation of the effect of static and impact loadings on bridge floor slabs of a different type see PUBLIC ROADS, vol. 8, No. 8, October, 1927, "Tests of the Delaware River Bridge Floor Slabs," by George W. Davis.

Va. The maximum size of the coarse aggregate was three-fourths inch.

The concrete was mixed in a power-driven mixer for two minutes, wheeled to the slab and vibrated into place with an electrically operated vibrator, as shown in Figure 3. After the concrete had been vibrated into place the surface was struck off with a wooden screed and roughly floated with a wooden float. It was then covered with wet burlap until the next morning, when the burlap was replaced by a 6-inch layer of straw. The straw was kept wet for 10 days, after which the slab was exposed to the air until the beginning of the tests at the age of 40 days.

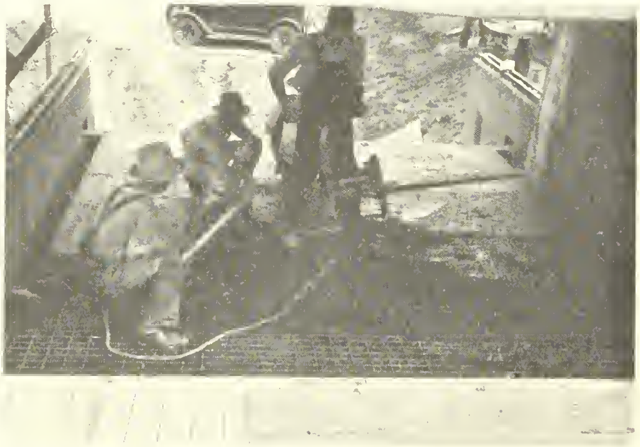


Figure 3.—Completed Steel Mat of First Slab Being Filled with Concrete by Means of Electric Vibrator

The concrete, as it came from the mixer had an average slump of 3¼ inches. Five 6 by 12 inch compression cylinders made at this time and cured in a damp room showed an average compressive strength at 28 days of 3,360 pounds per square inch.

The second slab was 12 feet, 7½ inches by 15 feet, 4 inches in area and 3 inches thick. As with the first slab, it was made up of five factory assembled mats 12 feet, 7½ inches long and 36 inches wide. In this mat the T-bars were 3 inches wide on the base and twelve were used to a mat. The bases of adjacent T-bars were held together at four places (in the 12 foot, 7½ inch length) by electric welds 2 inches long.

The stems of the T-bars in the second slab were tied together at 4-inch intervals by transverse ⅝-inch, half-round bars, welded with the flat side up, into the upper edge of the stems. (See fig. 4). At the edges of the 3-foot mat sections, the ends of these bars were bent aside and downward so that when embedded in concrete they served to unite adjacent sections.

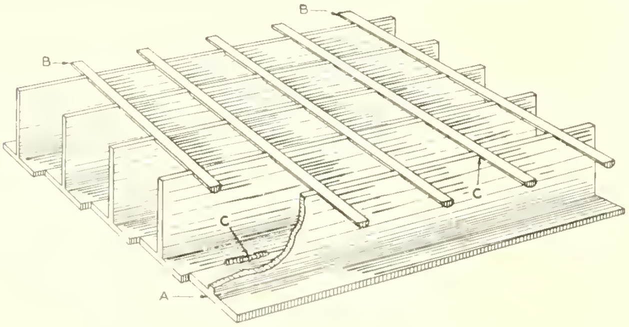


Figure 4.—Assembly of Steel Section of Second Slab

The appearance of the five sections of mat, when placed on the stringers, as in the case of the first slab, and welded, is shown in Figure 5.

The concrete which was used to fill the mat of the second slab was of the same proportions and aggregates as that used in the construction of the first slab. The cement, however, was of a different brand. A water-cement ratio of 0.82 was used, resulting in an average slump of 2¾ inches. Six test cylinders cured in a damp room showed an average compressive strength at 28 days of 5,030 pounds per square inch.

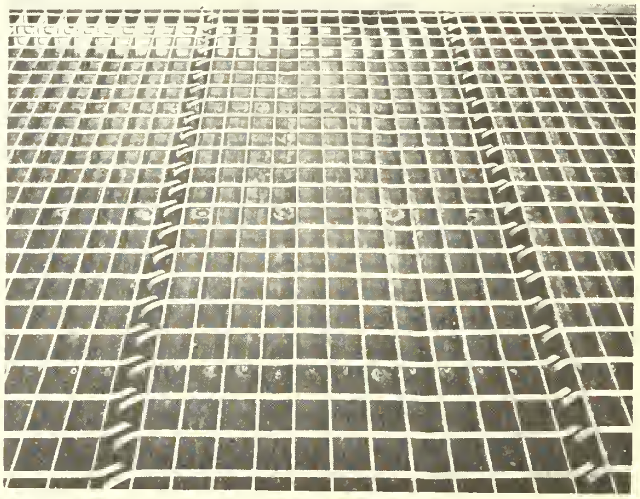


Figure 5.—Completed Mat Used in Construction of Second Slab Before Being Filled With Concrete. Note Number and Position of Welds and also Method Used for Uniting Edges of Individual Mats

The mixing, placing, finishing, and curing operations for the concrete of the second slab were the same as those used in the construction of the first slab.

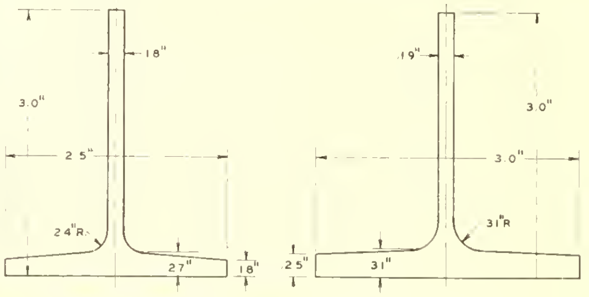


Figure 6.—Cross Sections of T-Bars Used in the Two Slabs

Figure 6 shows sections of the T-bars used in each slab. The calculated weight of the slabs is given in the following tabulation:

Weight per square foot of floor, in pounds

	First slab	Second slab
T-bars	18.0	20.0
Cross bars	1.3	1.6
Concrete	31.6	30.9
Total	50.9	52.5

In each test the slab was supported by four stringers set on 4-foot centers. These stringers were 18-inch 55-pound I-beams, rivet connected at their ends to

24-inch 80-pound **I**-beams. The floor beams were supported at their ends, in turn, by the concrete abutment walls, being held in their seats by anchor bolts. The distance between the floor beams was 16 feet and their span was 15 feet.

DESCRIPTION OF THE TEST PROGRAM

Both slabs were subjected to exactly the same program of tests. Since the object of the investigation was primarily to determine the effect of impact, the major part of the program was devoted to loadings intended to develop this information, and these loadings were applied at the center of the slab. Certain additional information was desired by Allegheny County, and the edge and 2-point loadings were included to provide these data. The detailed program follows:

1. Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at—
   - *a.* The center of the slab;
   - *b.* A point 1 foot from the edge of the slab and midway between the two center stringers;
   - *c.* Two points equidistant from the center of the slab along the axis transverse to the stringers, each point bearing half the load.

For all of the above loadings the deflections of both the slab and **I**-beams were determined and strains were measured both in the **T**-bars and in the two center stringers at the points shown on Figures 7 and 8.

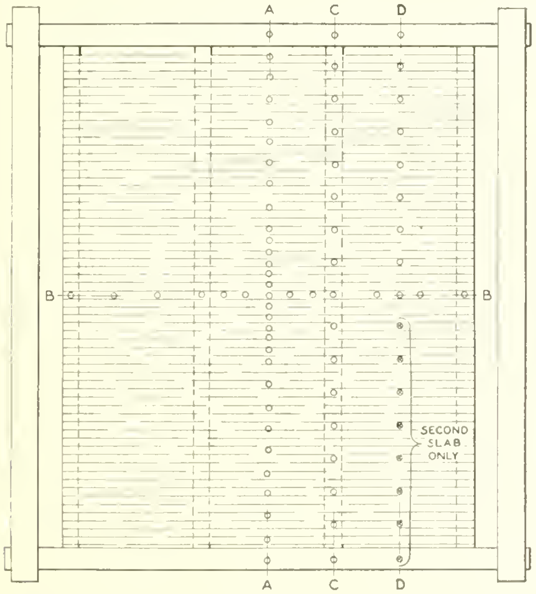


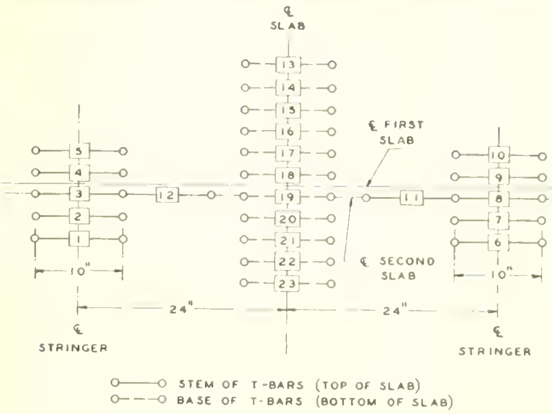
Figure 7.—Plan of Slab Showing Deflection Points

The spread or separation of the adjoining edges of the bases of the two **T**-bars directly under the load was measured for all of the loads applied at the center of the slab and for the 2-point loading.

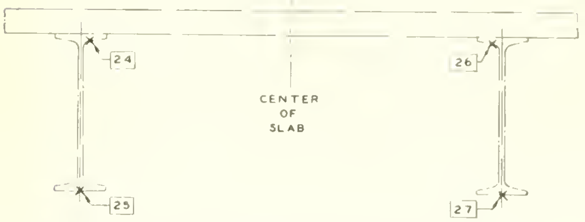
2. A series of 1,000 impact blows simulating those delivered by the wheel of a heavy motor truck was applied at the center point of the slab.

3. Static loads as in 1, *a* were applied and the same measurements of deflection and strain were made.

4. A second series of 1,000 impact blows was applied at the center of the slab, the average maximum force developed being about 11 per cent greater than on the first series of impact.



PLAN SHOWING LOCATION OF STRAIN GAGE POINTS IN THE T-BARS



SECTION SHOWING LOCATION OF STRAIN GAGE POINTS AT MIDPOINT OF THE STRINGERS

Figure 8.—Locations of Strain-Gage Points

5. Static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. Deflection and strain measurements were made as in the case of the previous static loadings.

6. A third series of impacts was applied at the center of the slab, the maximum force being very nearly the same as in the second impact series.

7. A fourth and last series of static loads was applied at the center of the slab, the load magnitudes, deflection, and strain measurements being the same as in 1, *a*.

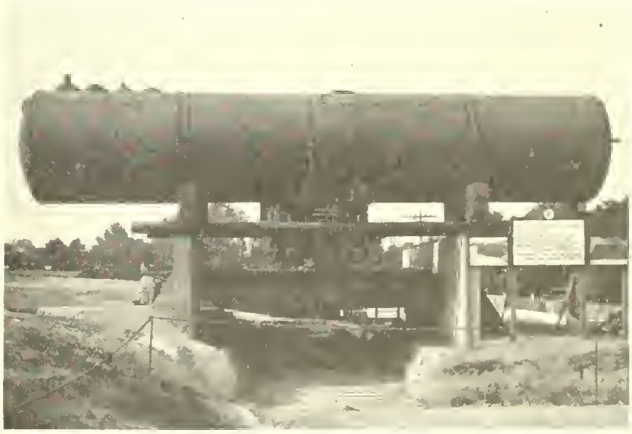


Figure 9.—General Arrangement of Test Slab, Supporting Walls, and Tank Used for the Application of the Static Loads

DESCRIPTION OF THE TESTING EQUIPMENT

Because of the large loads necessary for the static loading of these slabs and because of the alternate application of static and impact loads, a special set-up was designed to permit the execution of the program with facility.

The static loads were applied to the slabs with a hydraulic jack. This jack reacted against a large cylindrical steel tank containing water. The concrete walls, to which the ends of the floor beams were fastened, were extended 2 feet above the surface of the slab and supported the tank at an elevation sufficient to permit the installation of the jacking apparatus beneath it. On the tops of the two walls steel track was fastened, on which rested the rollers which supported the tank. The walls were made long enough to allow the tank to be moved back out of the way when impact tests were being made. The tank when filled with water weighed approximately 60,000 pounds. Figure 9 shows the general arrangement of the tank, slab, and supporting walls.

Attached to the under side of the tank was a beam of heat-treated steel which took the thrust of the jack. The load-deflection rate of this beam was known. To determine the magnitude of the loads being imposed on the slab, the deflection of the beam was measured with a micrometer dial. At the lower end of the jack a bearing block shod with segments of a solid rubber tire was provided. The details of this loading equipment are shown in Figure 10.

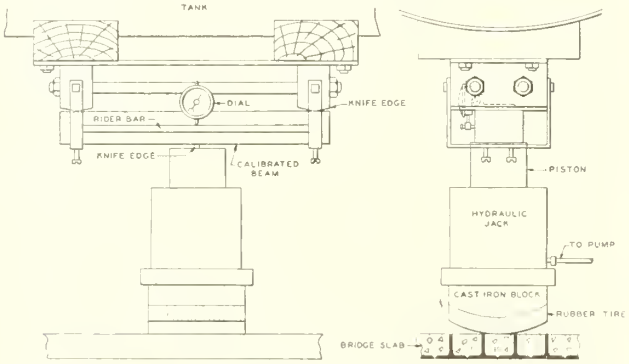


FIGURE 10.—APPARATUS USED IN APPLICATION AND MEASUREMENT OF LOAD

DEFLECTION MEASUREMENTS

The deflection of the slab and of the supporting I-beams was determined by measuring accurately changes in distance between gage points drilled into the lower surface of the slab or I-beams and corresponding gage points located directly beneath these in strips of steel fastened to concrete bases cast in the ground below the slab.

Changes in distance were measured with micrometer dials, reading in thousandths of an inch, fixed to the ends of wooden staffs. The staff and the dial were provided with conical steel points which fitted into the respective gage points.

Two pairs of gage points were set in the abutment walls to serve as a standard to check the length of the staffs at any time during a series of measurements.

The soil between the abutment walls was excavated to a lower level to give headroom under the slab for the measuring operations and to provide undisturbed earth as a foundation for the concrete bases carrying the deflection gage points. Figure 11 gives a good idea of the arrangements for the deflection measurements.

All of the deflection measurements were made with two movable staffs of different length, one being used under the stringers and the other under the T-bars of the slab proper. These staffs were moved from point to point after the desired load was on the slab. The deflection of the floor beams was measured by similar staffs which, however, were left in position and not moved from point to point. The positions of all deflection points are shown in Figure 7.

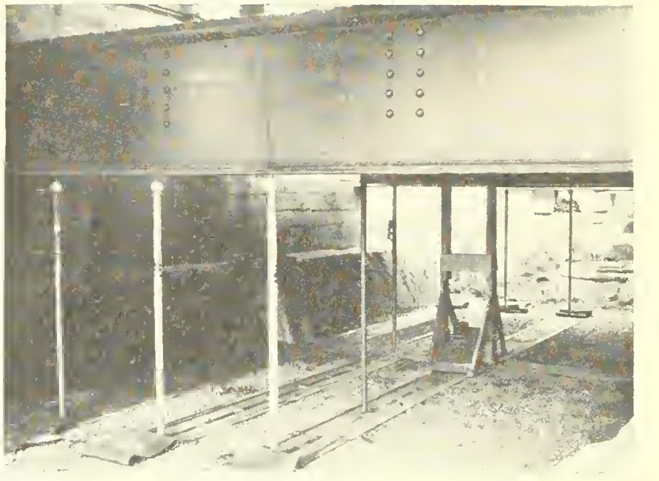


FIGURE 11.—LOWER DEFLECTION POINTS AND MICROMETER DIALS USED FOR THE MEASUREMENTS

STRAIN MEASUREMENTS

In these tests the strain measurements were confined to the steel and the strain gage positions were located in the regions where relatively large moments would be expected under the center loading. Figure 8 shows the locations at which strains were measured.

For making the measurements a 10-inch Whittemore strain gage was used. This gage was developed in connection with the Arch Dam Investigation conducted under the direction of the Engineering Foundation, and has been described in detail elsewhere.[3]

Briefly, it consists of parallel side bars of nickel steel connected at their ends by steel spring fulcrum plates which maintain the alignment of the bars. Two pointed legs for insertion in the gage holes are provided at opposite ends of the gage, one leg being attached to each side bar. The relative displacement of the side bars longitudinally is indicated by a micrometer dial reading directly in ten-thousandths of an inch. Corrections for temperature changes are made by reference to an unstressed steel bar, in the usual way.

Strain gage holes were drilled with a No. 56 twist drill and the burr removed with a special reamer.

SPREAD OF THE T-BARS

In order to determine the amount of separation that occurred between adjacent T-bars directly beneath the loaded area, a small micrometer dial was attached to the bottom of the base of one T-bar and the stem of this dial acted against a small stud in the adjacent T-bar.

IMPACT TESTS

Impact was applied by means of the portable impact machine developed by the Bureau of Public Roads and described in PUBLIC ROADS several years ago.[4] The method of test was the same as that used in the tests of the Delaware River Bridge floor slabs, referred to above.

In these tests the acceleration of the unsprung mass was measured by means of an accelerometer of the single-element contact type.[5]

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[3] Instruments, vol. 1, No. 6, p. 299, June, 1928, Whittemore Strain Gage, by H. L. Whittemore.

[4] PUBLIC ROADS, vol. 5, No. 2, April, 1924, Impact Tests on Concrete Pavement Slabs, by Leslie W. Teller.

[5] For discussions of the use of accelerometers for the measurement of motor vehicle impact see the following publications: (1) Proceedings, American Society for Testing Materials, 1923, An Accelerometer for Measuring Impact, by E. B. Smith; (2) PUBLIC ROADS, vol. 5, No. 10, December, 1924, Accurate Accelerometers Developed by the Bureau of Public Roads, by L. W. Teller; (3) PUBLIC ROADS, vol. 11, No. 5, July, 1930, Calibration of Accelerometers for Use in Motor Truck Impact Tests, by J. A. Buchanan and G. P. St. Clair.

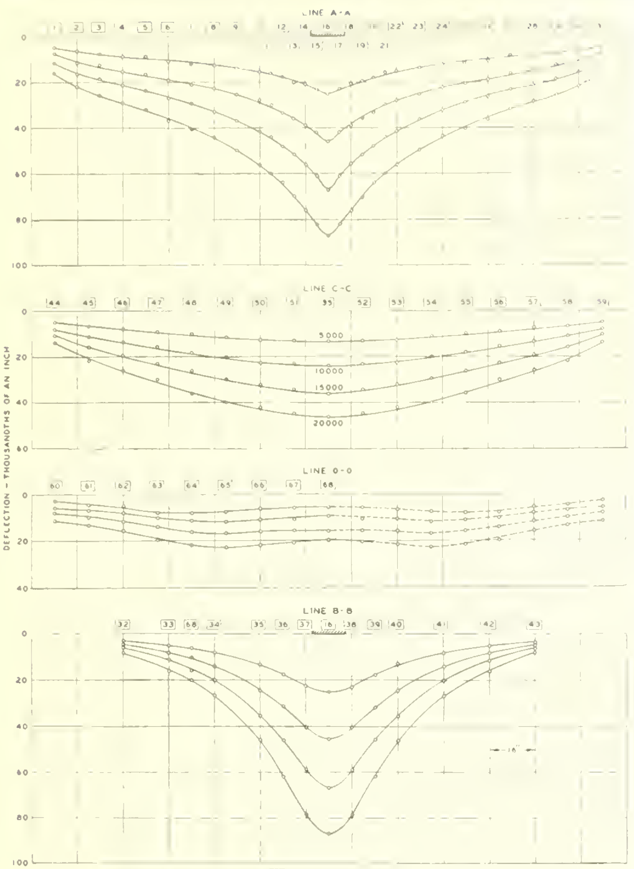


FIGURE 12.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING BEFORE IMPACT

DETAILED DESCRIPTION OF THE TESTING OF THE FIRST SLAB

Static loads at the center.—Before any deflection or strain data were taken, several preliminary static loads were applied to the center of the slab. The magnitude of each successive preliminary load was increased until a load of 20,000 pounds had been applied. The object of these loadings was to place the structure in a cyclic state or, in other words, in a condition such that successive applications of a given load would produce the same structural effects. These loadings were also utilized for necessary adjustments of the loading and measuring apparatus.

Initial static loading.—Following the application of the preliminary loads, two series of initial test loadings were applied to the center of the slab. Each series consisted of loads of 5,000, 10,000, 15,000, and 20,000 pounds, and the two series were applied on successive days with a recovery period of about 18 hours between. Under each load of these two initial series complete deflection and strain measurements were made.

The zero readings for deflection and strain—i. e., measurements for the unloaded condition—were made with the entire loading apparatus lifted clear of the slab and in applying all of the static loads due allowance was made for the dead weight of all of the loading equipment below the calibrated beam.

In order to establish beyond question the data which were to serve as a basis of comparison for the stiffness of the slab before impact—i. e., the deflections and strains under the center loading—the third series of initial loadings were applied several days after the first two. The data obtained at this time consisted of one set of measurements for deflection at all of the deflection points on the slab for loads of 5,000 and 20,000 pounds and a series of measurements of deflection at deflection point 16 and strain measurements at strain gage point 15 for 10 applications each of 5,000, 10,000, 15,000, and 20,000 pound loads, made at approximately half-hour intervals.

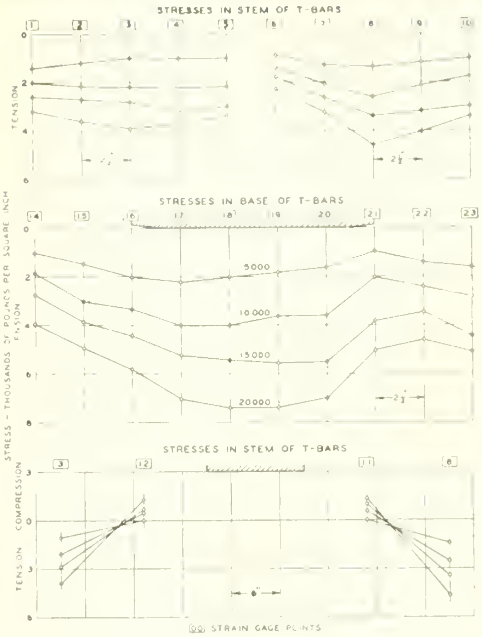


FIGURE 13.—STRESSES IN FIRST SLAB CAUSED BY STATIC LOADS BEFORE IMPACT

The data obtained under successive applications of a given load agreed within very close limits and the mean values of all of the measurements made under each of the several loads were taken to express the load-deflection and load-strain conditions in the slab before impact was applied. These mean deflection curves are shown in Figure 12 and the strain data for the T-bars are shown in Figure 13 as stresses, computed with an assumed value for the modulus of elasticity of 30,000,000 pounds per square inch.

As the load was applied at the center of the slab, the deflection of points symmetrically spaced with respect to its axes should be equal. Actually this was found to be true, the differences in the measured deflections of symmetrically placed points being negligible. Therefore, throughout this report, in plotting the deflection curves for the various loads and lines of points the measured deflections of symmetrical points have been averaged.

With the load at the center point the separation or spread which occurred between the adjacent edges of the bases of the two T-bars directly underneath the load was measured, as previously described. These measurements were made for loads 5,000, 10,000, 15,000, and 20,000 pounds on each of two successive days. The two sets of values agreed very closely and are averaged in the following tabulation:

Load in pounds	Spread in inches
5,000	0.0018
10,000	.0035
15,000	.0050
20,000	.0064

Static loads at the edge.—Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at a point 1 foot from the edge of the slab on the axis of the slab parallel to the stringers.

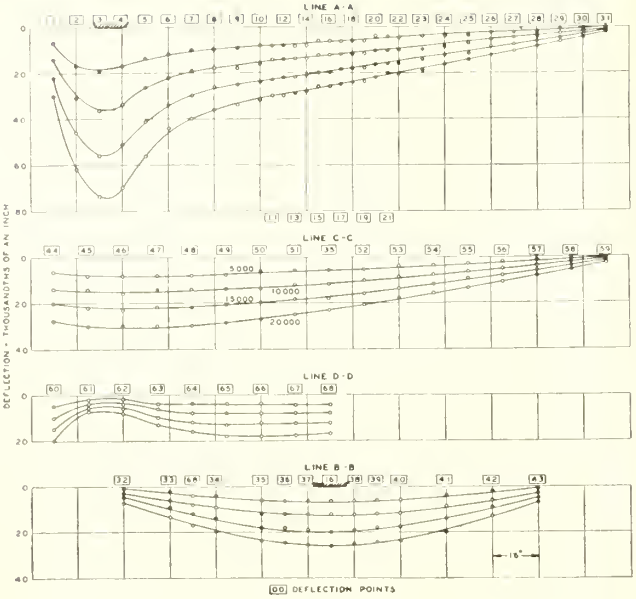


FIGURE 14.—DEFLECTIONS OF FIRST SLAB UNDER EDGE LOADING BEFORE IMPACT

The deflection curves for the slab and for the stringers nearest the load are shown in Figure 14. Two sets of measurements of deflection were made for two applications of each of the four loads on the same day. The two sets of values agreed with each other within close limits and the values shown are for the means of the two measurements.

No strain measurements of any significance were obtained under this loading since the strain gage locations were in areas of small stress. The indicated stress at the midpoint of the two center stringers was 3,300 pounds per square inch in the bottom flange under the 20,000 pound load.

The 2-point static load.—Another special loading was that applied equally on two tire segments set equidistant from the midpoint of the slab along the axis normal to the stringers. These tire segments were 72 inches apart, center to center.

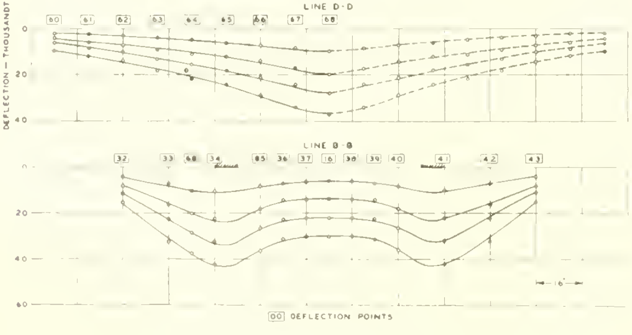


FIGURE 15.—DEFLECTIONS OF FIRST SLAB UNDER TWO-POINT LOADING BEFORE IMPACT

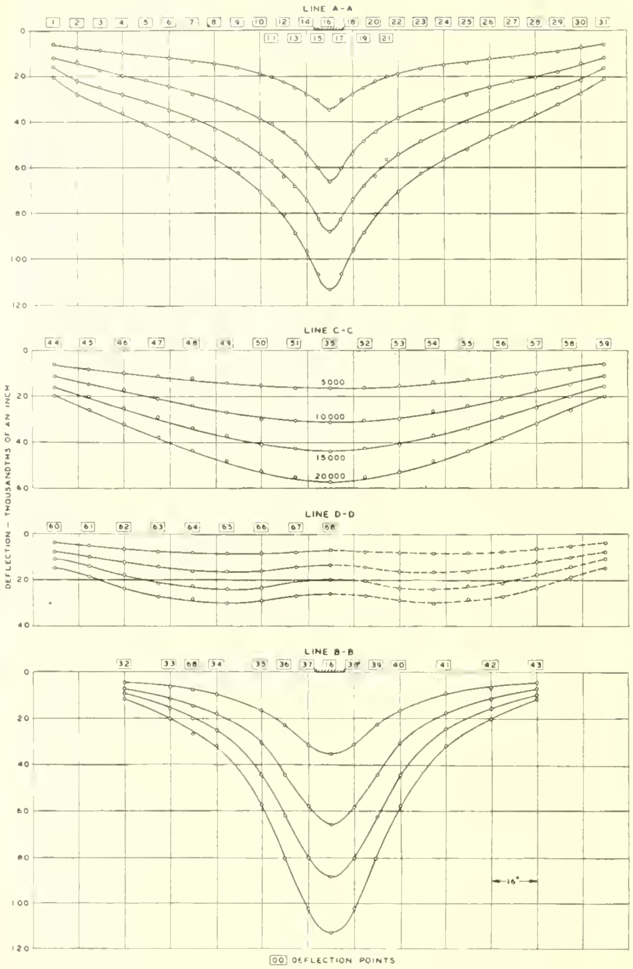


FIGURE 16.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER FIRST IMPACT

The deflection curves for loads of 5,000, 10,000, 15,000, and 20,000 pounds applied in this manner are shown in Figure 15. As in the case of the edge loading, the curves represent mean values from two sets of measurements. The strains produced at the strain gage locations shown in Figure 8 were too small to be determined with the strain gage used in these tests, except at the midpoint of the lower flange of the center stringers where a maximum stress of about 3,000 pounds per square inch was indicated.

The spread of the bases of the two T-bars at the midpoint of the slab was also measured under the 2-point loading and was found to be very small, as shown in the following tabulation:

Total load, in pounds	Spread, in inches
5,000	0.0002
10,000	.0004
15,000	.0006
20,000	.0009

Application of the first impact.—In all of the impacts applied to the bridge floor slabs in these tests, the conditions selected were such that very severe impact was produced. The conditions approximated those of a loaded 5-ton truck, equipped with solid tires, the unsprung weight being 1,967 pounds and the spring deflection being that which would correspond to a sprung load of about 6,000 pounds. The tire used on

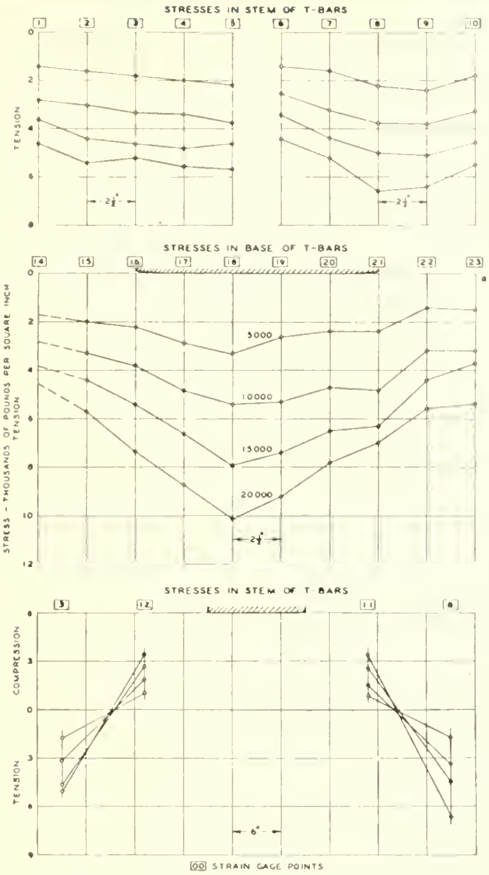


FIGURE 17.—STRESSES IN FIRST SLAB CAUSED BY STATIC LOADS AFTER FIRST IMPACT

the wheel of the impact machine in the tests of the first slab was a 36 by 6-inch solid rubber truck tire. With these conditions the first series of 1,000 drops of the impact machine were applied. A free fall of 0.25 inch was used. The measured accelerations varied between 310 and 354 feet per second per second, the average impact force for this series being 28,200 pounds. The rate at which the impact was applied was approximately seven blows per minute.

Careful observation of the slab during the application of the first impact and examination afterwards showed no visible effect of the severe hammering. The thin grout or laitance between the edges of the T-bars on the bottom of the slab remained in place even directly under the point of impact and no visible structural cracks appeared on the upper surface of the slab.

After the completion of these tests the impact machine was removed from the slab, the static loading equipment was replaced, and a second series of static loads applied at the center of the slab in accordance with the program.

Static loading after the first impact.—Loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at the center of the slab and complete deflection and strain measurements were made for each load. Two sets of tests were made and the averaged data for the deflections are shown in Figure 16 and for the strains in Figure 17.

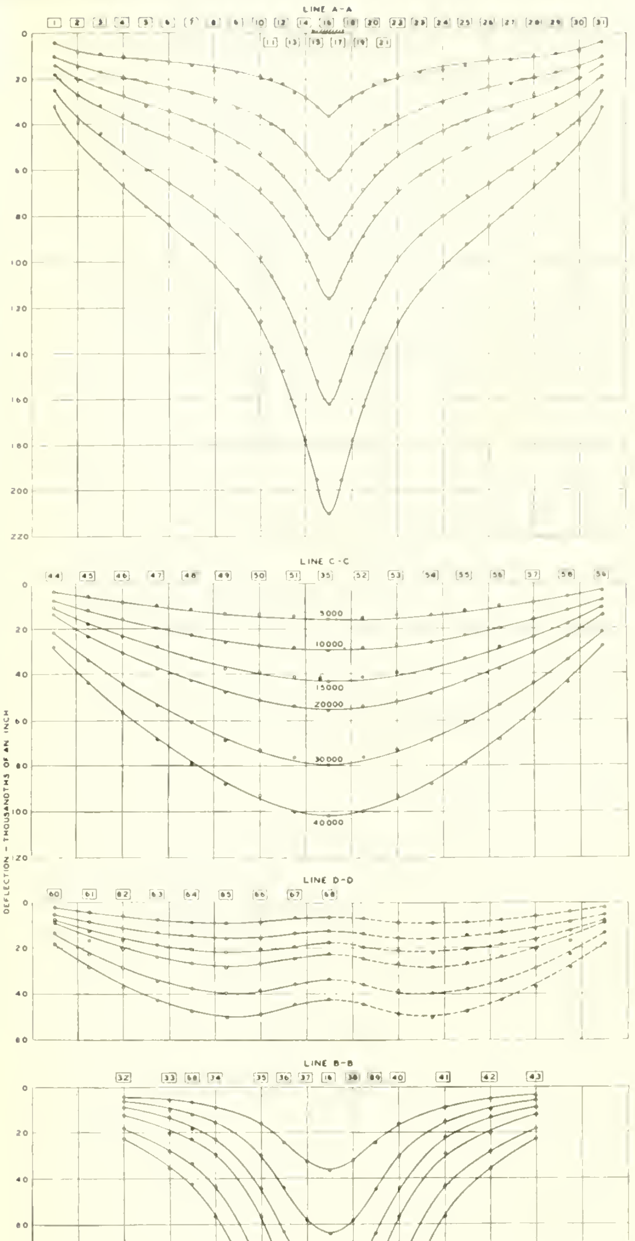


FIGURE 18.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER SECOND IMPACT

If these data are compared with the corresponding data obtained before the impact was applied it will be noted that for a given load there was a definite increase in the magnitudes of the deflections and of the stresses in the T-bars after the first series of impacts. For example, under a 20,000-pound static load at deflection point 16 (the center of the slab) an increase of from 0.087 to 0.113 inch (30 per cent) occurred. Similarly, the maximum stresses in the T-bars increased after impact from 1,650 to 6,600 pounds per square inch (42 per cent) over the stringers and from 7,350 to 10,200 pounds per square inch (39 per cent) directly underneath the load. Strain measurements in the top of the T-bars between the load and the stringers showed an increase in compressive stress of from 1,350 to 3,450 pounds per square inch (about 155 per cent) after the impact.

Application of the second impact.—The second series of 1,000 drops of the wheel of the impact machine was applied at the same point and in the same manner as the first series, except that the free fall of the wheel before the tire made contact with the slab was increased from 0.25 to 0.63 inch in order to produce an impact force of greater magnitude. The effect of this was to increase the range of measured accelerations from 310–354 to 378–426 feet per second per second. The average maximum impact reaction for the second series of 1,000 drops was 31,300 pounds.

This impact produced no visible change in the appearance of either surface of the slab.

Static loading after the second impact.—Upon the completion of the second series of impact tests, the machine was removed and static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. These loads were repeated, complete deflection and strain measurements being made for each loading. The deflection data are shown in Figure 18. The corresponding strain data, expressed as unit stresses, for all of the strain gage positions shown in Figure 8 are given in Table 1.

TABLE 1.—*Unit stresses in* T-*bars of first slab under center loading after second impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	1,650	----	2,400	2,100	2,250	1,350	1,800	1,950	2,400	1,950	750	1,050
10,000	2,700	----	3,600	3,450	3,750	2,550	3,300	3,450	4,200	3,750	2,100	2,250
15,000	3,450	----	4,800	4,500	4,800	3,450	4,800	5,400	5,700	4,950	3,150	2,850
20,000	4,500	----	5,700	5,550	6,150	3,900	5,550	6,300	6,600	6,000	4,350	3,750
30,000	6,300	----	8,100	8,100	8,700	5,700	7,800	8,700	9,600	8,700	5,400	5,400
40,000	8,250	----	10,800	10,500	10,500	7,500	9,300	11,550	10,950	10,650	6,900	6,150

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	------	1,050	1,650	1,950	2,700	2,850	2,100	2,400	1,800	900
10,000	------	2,700	3,450	4,200	5,700	5,250	4,500	4,050	2,850	1,650
15,000	------	4,200	5,100	5,700	8,100	7,050	5,850	5,100	4,500	3,300
20,000	------	5,850	6,600	7,950	10,200	9,450	7,800	7,050	5,400	4,050
30,000	------	8,400	9,600	11,700	15,000	14,100	12,000	9,900	8,400	6,300
40,000	------	11,100	13,200	15,300	19,800	18,450	15,000	12,150	10,650	9,000

A comparison of the data for the four lowest loads with those obtained under the same loads before the second impact show that there was no appreciable increase in either deflections or stresses. The data obtained for the 30,000 and 40,000 pound loads indicate that the action of the slab under these loads was elastic and no apparent damage to the slab was observed.

Application of the third impact.—The third impact consisted of a series of 1,000 drops, duplicating as closely as possible the second series. The same free fall was used and the average impact reaction for this series was 31,900 pounds.

Static loading after the third impact.—The final static tests consisted of the application of loads of 5,000,

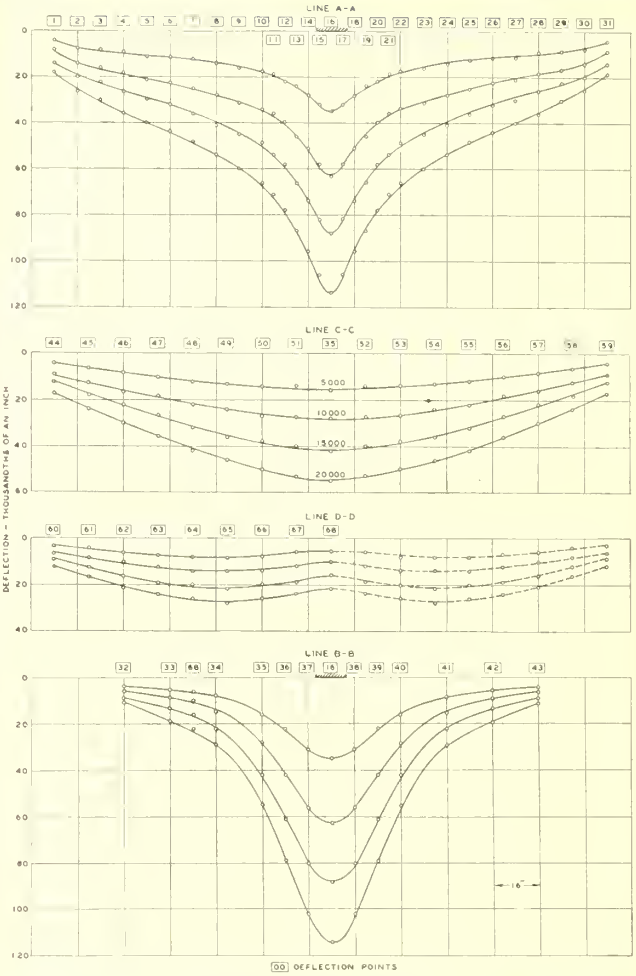


FIGURE 19.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER THIRD IMPACT

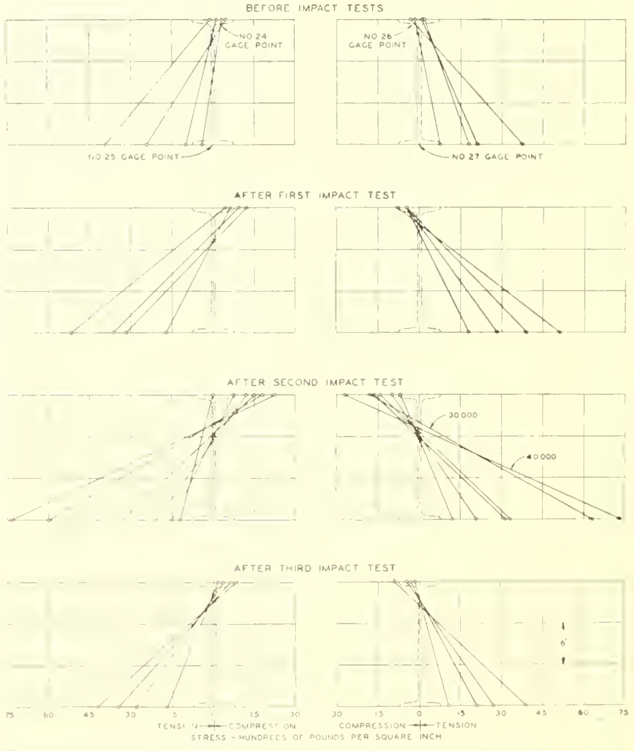


FIGURE 20.—STRESSES IN FIRST SLAB AT MIDPOINT OF STRINGERS UNDER STATIC LOADS OF 5,000, 10,000, 15,000, AND 20,000 POUNDS

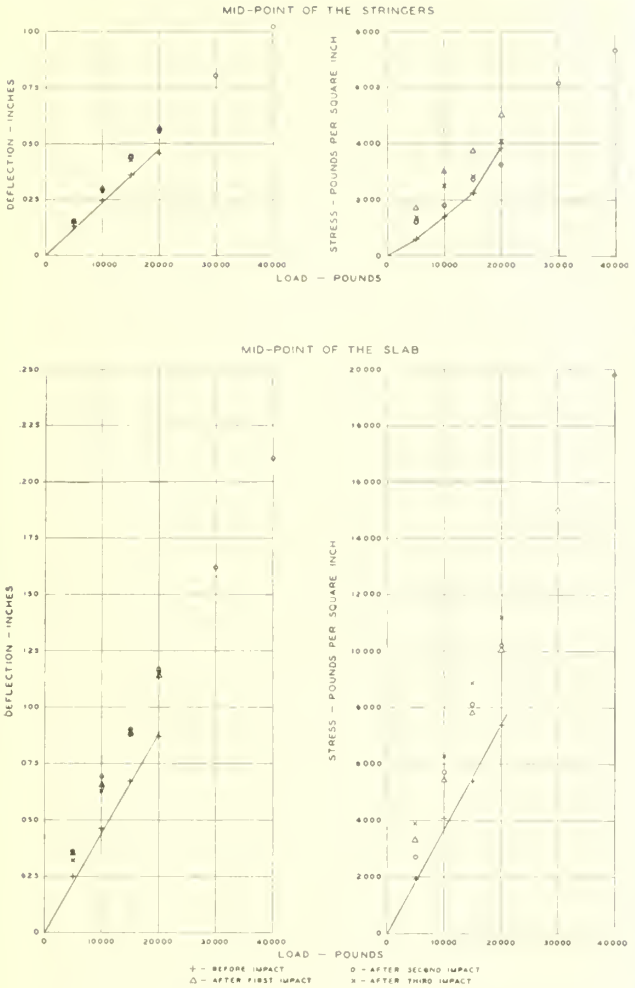


FIGURE 21.—COMPARISON OF DEFLECTIONS AND STRESSES IN FIRST SLAB BEFORE AND AFTER IMPACT

10,000, 15,000, and 20,000 pounds at the center of the slab with complete deflection and strain measurements for each loading. The deflection data for these loadings are shown in Figure 19 and the stress data in Table 2.

When these data are compared with those obtained for the same loads before the application of the third impact, it is evident that there is no significant difference.

Stresses in the stringers.—Mention has been made of the fact that strain measurements were made at the mid-points of the two stringers nearest the center line of the slab. The strains measured were those in the upper and lower flanges of the I-beams at the positions shown in Figure 8.

The strain data obtained at these points for all of the static loadings applied to the slab are shown in Figure 20 expressed as unit stresses. It is indicated that the stress in the upper flanges was extremely small and that the stress in the lower flanges did not exceed 5,100 pounds per square inch (except for the 30,000 and 40,000 pound loads). The position of the neutral axis of the stringers was not very definitely determined because of the small deformations measured in the upper flanges of the I-beams, but generally it appears to be from 12 to 15 inches above the lower flange.

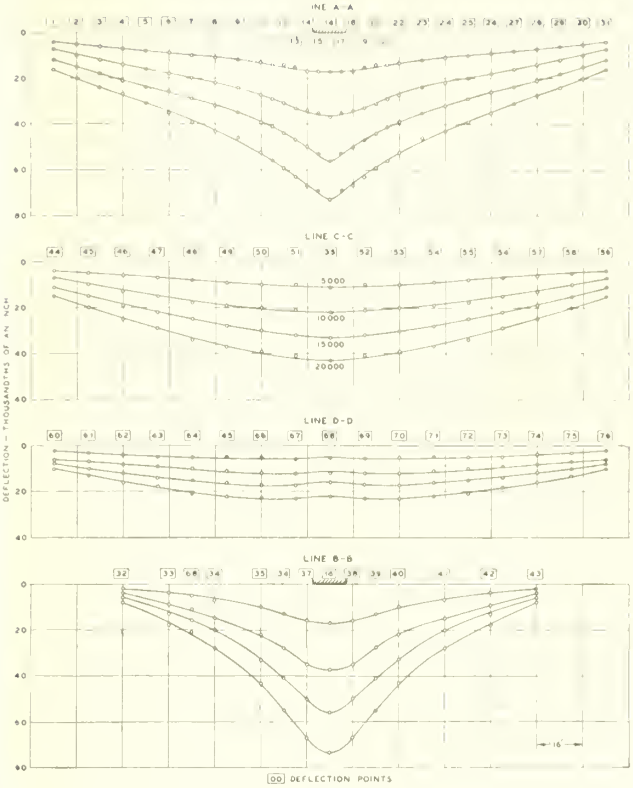


FIGURE 22.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING BEFORE IMPACT

TABLE 2.—*Unit stresses in T-bars of first slab under center loading after third impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	1,950	----	2,400	2,250	2,250	1,350	1,950	2,250	2,550	2,400	750	1,050
10,000	3,300	---	3,750	3,600	4,200	2,700	3,450	4,200	4,050	3,750	1,800	2,250
15,000	4,200	--	5,100	5,100	5,100	3,900	4,950	6,000	5,700	5,250	2,550	3,150
20,000	4,950	-	6,300	6,150	6,300	6,000	6,150	7,200	6,900	6,600	3,750	1,050

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	------	1,350	2,550	2,700	3,900	3,000	2,550	2,250	1,500	1,650
10,000	------	3,300	3,600	4,650	6,300	5,850	4,950	4,650	3,000	2,400
15,000	------	4,650	5,700	6,450	8,850	7,650	6,750	6,150	4,050	4,200
20,000	------	5,700	7,350	8,250	11,250	10,050	8,400	7,350	5,850	5,250

In Figure 21 a comparison is made of the structural action of the stringers before and after impact and of the T-bars at the center of the slab before and after impact. For this purpose the load-deflection and the load-stress relations for the mid-point of the stringers and for the base of the T-bar at the mid-point of the slab were selected. On this graph the values obtained before impact are connected with a solid line and the observed values after the several impact tests are simply plotted points.

DETAILED DESCRIPTION OF THE TESTING OF THE SECOND SLAB

Static loads at the center.—In testing the second slab the general procedure was exactly the same as that followed in the testing of the first slab. The same loading program was followed throughout except for slight variations in the magnitude of the impact forces applied.

Following a series of preliminary loadings, as in the case of the first slab, three series of initial static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at the center of the slab on three different days and complete deflection and strain measurements were made for each. The mean deflection curves obtained are shown in Figure 22 and the strain data expressed as unit stresses are shown in Figure 23.

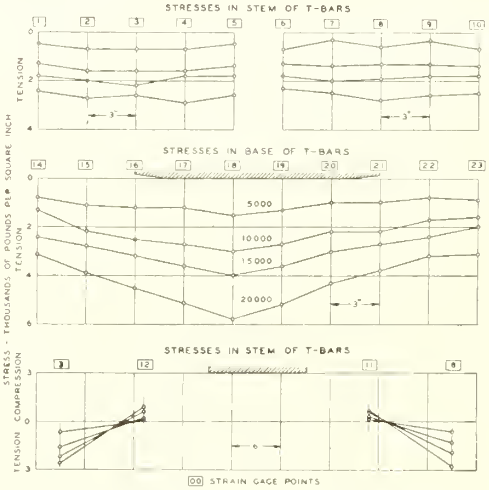


Figure 23.—Stresses in Second Slab Caused by Static Loads Before Impact

The spread between the bases of the two adjacent T-bars directly under the load was measured in the same manner as was used in the previous test and found to be very small, being only 0.003 inch for the 20,000-pound load.

Static loads at the edge.—Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied twice at a point 1 foot from the edge of the slab and on the axis of the slab parallel to the stringers.

The deflection curves in Figure 24 are the mean values obtained in these loadings. The only significant strain measurements obtained were at the centers of the stringers where stresses of about 1,000 pounds per square inch were indicated.

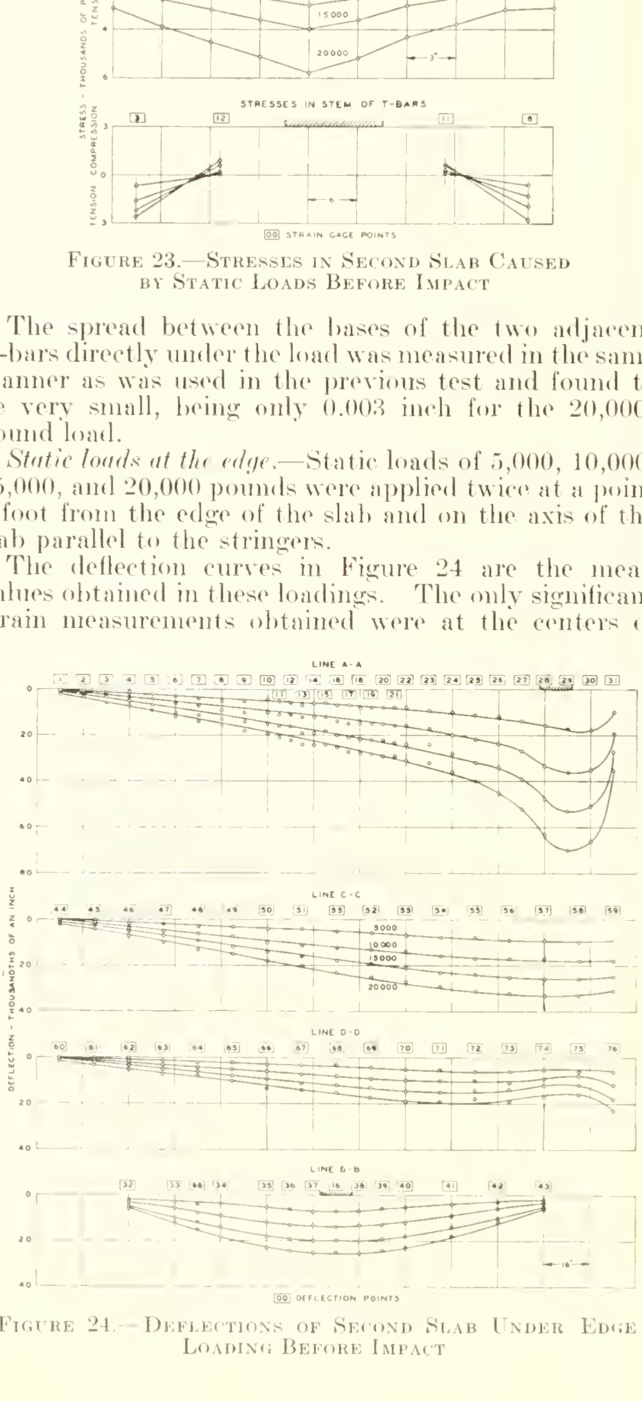


Figure 24.—Deflections of Second Slab Under Edge Loading Before Impact

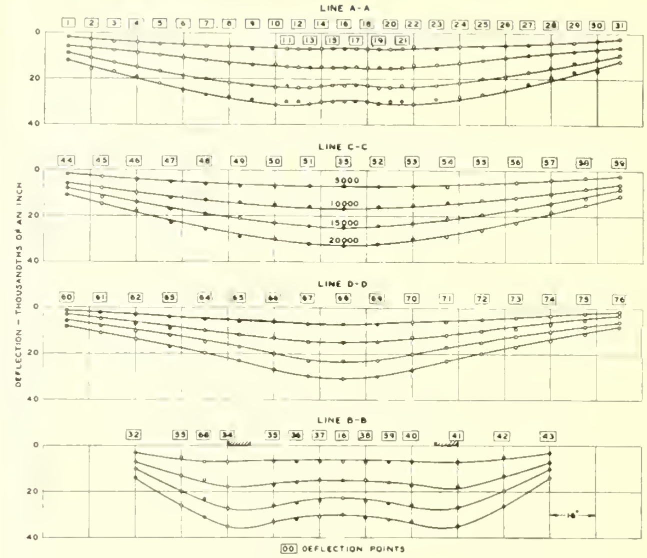


Figure 25.—Deflections of Second Slab Under Two-Point Loading Before Impact

The reversal of the arrangement of the deflection curves for this loading in the presentation of the data for the two slabs is due to the fact that opposite edges were used for the edge loading of the first and second slabs.

The 2-point static load.—This special loading was applied in exactly the same manner as in the testing of the first slab. The loads were applied twice and the mean values of the measured deflections are shown in Figure 25. The strains produced by this loading at the strain gage positions shown in Figure 8 were too small to be measured with any accuracy except those at the mid-point of the stringers, where stresses of slightly less than 3,000 pounds per square inch were indicated.

There was no measurable spread between the T-bars at the center of the slab under the 2-point loading.

Application of the first impact.—In applying impact to the second slab the apparatus and procedure were the same as those used in the testing of the first slab. The average magnitude of the impact reaction in this first series of 1,000 drops was 27,300 pounds.

Following the application of this impact the testing machine was removed and the slab given a careful inspection. There was no visible effect of this impact on either the upper or lower surfaces of the slab. After this examination the static loading equipment was again placed on the slab.

Static loading after the first impact.—In accordance with the program, static loads of 5,000, 10,000, 15,000, and 20,000 pounds were next applied at the center point of the slab. Each load was applied twice and complete deflection and strain measurements were made for each load. The mean deflection data are shown in Figure 26 and the strain data, expressed as unit stresses, are shown in Figure 27.

If a comparison is made between these data and thos given in the corresponding figures before impact was applied (figs. 22 and 23), it will be noted that after the impact the maximum deflection of the slab for a give load was slightly greater than before impact. For example, the deflection of the center point under th 20,000-pound static load was 0.073 inch before impac and 0.077 inch after impact, an increase of about 5 pe cent. The comparison of the stress data will show tha

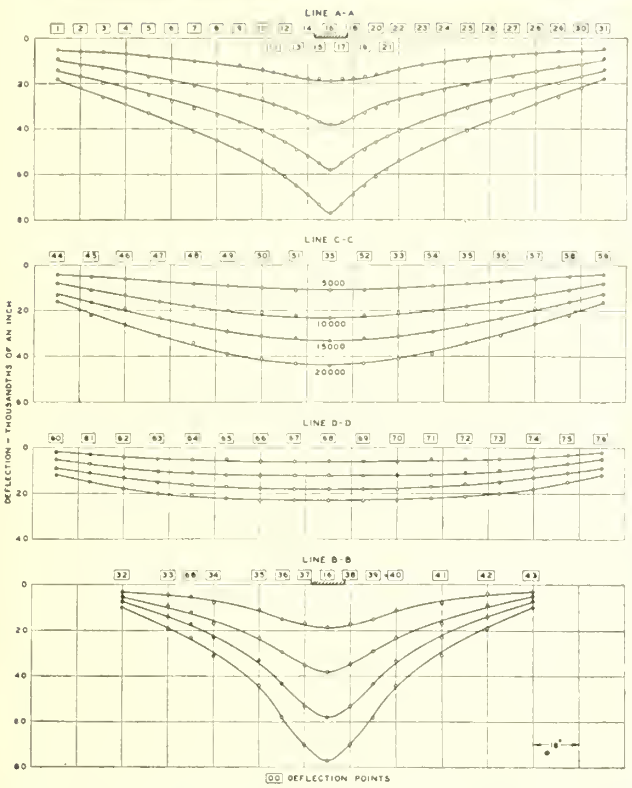


FIGURE 26.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING AFTER FIRST IMPACT

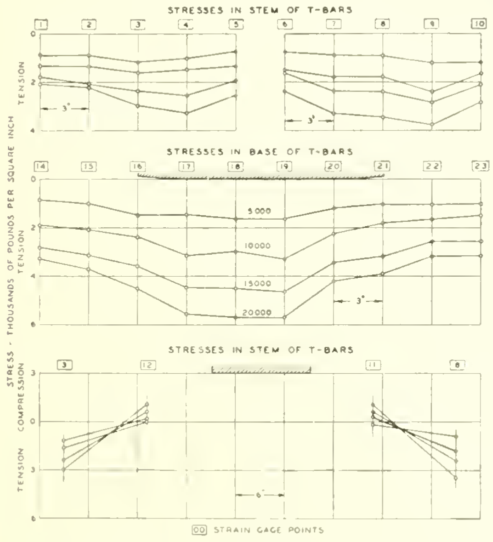


FIGURE 27.—STRESSES IN SECOND SLAB CAUSED BY STATIC LOADS AFTER FIRST IMPACT

there was some adjustment of the stresses in the T-bars near the load, but that this adjustment did not increase the maximum stress greatly for any of the loads. In the bases of the T-bars under the load there was apparently a small increase in the stresses due to the 15,000-pound load, while for the other three loads there was no increase. In the stems of the T-bars there was a small increase for most of the loads, but since the stress for the largest load was less than 4,000 pounds per square inch, this increase is of no particular importance except as another indication of a readjustment of the amount of load taken by the concrete and by the steel.

Application of the second impact.—In applying the second series of impact loads to the second slab the magnitude of the impact reaction was increased to an average value of 30,400 pounds by increasing slightly the height of free fall of the truck wheel of the impact machine. This corresponded to the value of 31,300 used in the second series on the first slab.

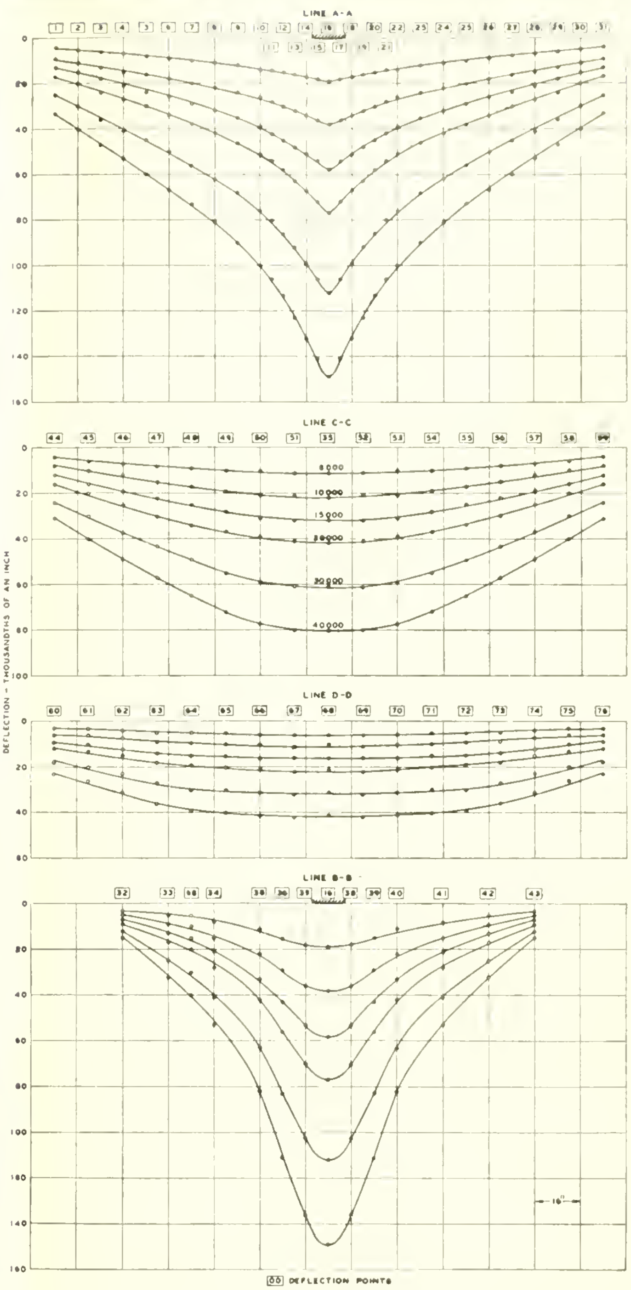


FIGURE 28.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING AFTER SECOND IMPACT

After the completion of 1,000 drops the impact machine was removed and the slab carefully inspected. There was no visible change in either surface of the slab.

Static loading after the second impact.—Static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. These loads were applied twice, complete deflection and strain measurements being made for each load. The mean deflection curves for these loadings are shown in Figure 28. The corresponding stress data for all of the strain gage positions are given in Table 3.

These data indicate that the application of the second impact did not produce any significant change

TABLE 3.—*Unit stresses in T-bars of second slab under center loading after second impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	900	900	1,200	1,050	600	750	750	1,050	600	1,200	300	[1] 450
10,000	1,350	1,500	1,950	1,950	1,500	1,050	1,350	1,800	1,500	1,800	450	[1] 150
15,000	1,950	2,100	2,850	2,700	1,950	1,800	1,800	2,700	2,100	2,400	750	150
20,000	2,850	2,850	3,750	3,750	3,000	2,550	2,700	3,750	3,150	3,150	1,050	300
30,000	4,200	4,650	5,550	5,550	4,500	3,750	4,650	5,550	4,650	4,800	1,650	600
40,000	5,700	6,150	7,500	7,650	6,300	5,550	6,450	7,650	6,900	6,300	2,400	1,050

BOTTOM OF BASES OF T-BARS

Load	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	1,050	1,200	1,200	1,650	1,950	1,350	1,350	1,200	1,200	1,200
10,000	1,950	2,100	2,400	2,700	2,850	2,400	2,850	1,800	1,350	1,650
15,000	2,250	3,000	3,150	3,750	4,350	3,750	3,300	2,850	2,100	2,100
20,000	3,300	4,050	4,500	5,250	5,400	4,950	4,800	3,600	2,850	2,850
30,000	5,700	6,750	7,950	8,700	9,150	8,550	7,650	6,000	5,700	4,950
40,000	7,050	8,400	9,900	11,400	12,150	11,100	9,900	8,100	7,200	6,750

[1] Tension.

in the behavior of the slab. The deflections were practically the same as those measured for corresponding loads after the application of the first impact and the load-deflection relation at the center of the slab is essentially a straight line up to and including the 40,000-pound load. The strains in the T-bars were practically unchanged for the four lower loads and the maximum stress indicated at any point was 12,150 pounds per square inch in the T-bar under the center of the 40,000-pound load.

Application of the third impact.—In this series the average impact reaction was 31,900 pounds, corresponding exactly to the value used in the application of the third series of impact loads to the first slab. After the completion of the series of 1,000 drops the slab was again carefully inspected but no visible effect of the impact was found.

Static loading after the third impact.—The final series of static tests comprised loads of 5,000, 10,000, 15,000, and 20,000 pounds applied at the center of the slab, complete deflection and strain measurements being made for each loading. The mean deflections are shown in Figure 29 and the stress data in Table 4.

A comparison of these data with those obtained under the same loads, before the third series of impact loadings was applied, shows that the third impact did not produce any significant change in the action of the slab.

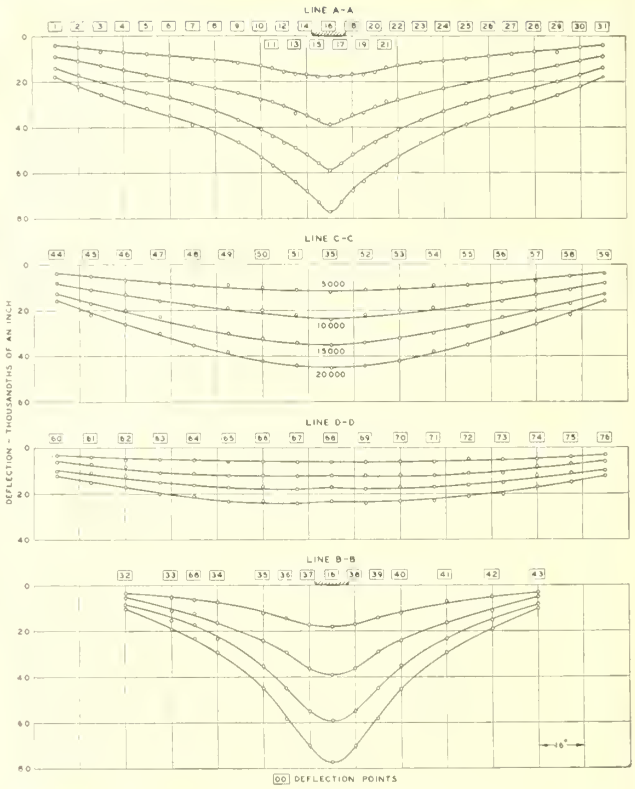


FIGURE 29.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING AFTER THIRD IMPACT

TABLE 4.—*Unit stresses in T-bars of second slab under center loading after third impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	900	600	900	600	750	750	750	750	900	750	[1] 150	[1] 300
10,000	1,650	1,350	1,500	1,500	1,350	1,200	1,650	1,650	1,650	1,200	0	0
15,000	2,100	2,250	2,250	2,400	2,250	2,250	2,400	2,550	2,850	2,250	450	300
20,000	2,850	3,150	3,450	3,300	2,850	3,150	3,150	3,600	3,450	2,700	450	150

BOTTOM OF BASES OF T-BARS

Load	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	1,050	900	1,200	1,200	1,500	1,350	900	1,050	600	300
10,000	1,650	1,800	2,100	2,550	2,700	2,850	1,800	1,950	1,650	1,050
15,000	2,550	2,850	3,450	4,050	4,050	3,900	3,000	2,700	2,550	1,650
20,000	3,600	3,900	4,650	5,250	5,100	5,100	4,050	3,450	3,300	2,400

[1] Tension.

Stresses in the stringers.—As in the case of the first slab, strain measurements were made at the mid-point of the two stringers nearest the center line of the slab. These strain data, expressed as unit stresses, are shown for all loads in Figure 30. It will be noted that there was practically no stress in the upper flanges and that even with the 40,000-pound load the stress in the lower flange did not exceed 7,000 pounds per square inch. The position of the neutral axis of these members is rather indefinite, because of the fact that the stresses in the upper flange of the stringers were so small as to be incapable of being measured accurately, but apparently it is near the lower surface of the slab.

Figure 31 shows the load-deflection and load-stress relations for the mid-point of the stringers and for the mid-point of the slab before and after the application of the impact. This figure is a companion graph to Figure 21, the values measured before impact being connected with a solid line and those observed after the three series of impact loadings being shown as plotted points. In these data there is an indication of the same increase in flexibility that was noted after impact was

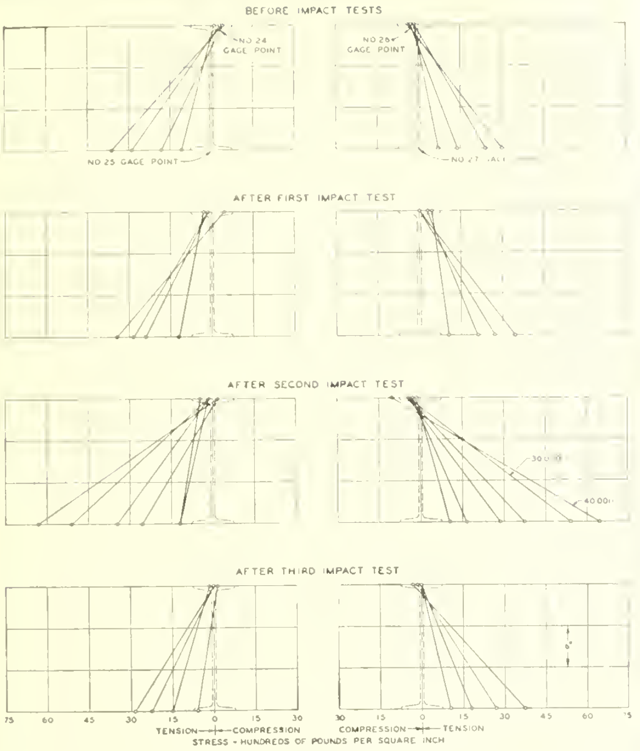


Figure 30.—Stresses in Second Slab at Midpoint of Stringers Under Static Loads of 5,000, 10,000, 15,000, and 20,000 Pounds

applied to the first slab but the magnitude of the increases both in deflection and in stress is very much less.

OTHER DATA OBTAINED

Deflection under impact.—At the conclusion of the scheduled tests on the second slab, advantage was taken of the opportunity to obtain some data on the relative deflections of the mid-point of the slab under impact and equivalent static loads.

The procedure used for measuring the slab deflection under impact was very simple. In the center of the base of the T-bar directly under the center of the load a short stylus point of hardened steel was attached. The stylus extended downward a short distance and the tip was bent over into a horizontal position. A substantial framework resting upon the earth beneath the slab supported, on edge, a small piece of plate glass, one side of which was coated with a smoke film. The smoked side of this glass plate was adjusted so that it just made contact with the horizontal stylus point. A deflection of the slab produced a short vertical trace in the smoke film. After each loading the glass plate was moved forward in guides to a new position, many records being made on one plate. The measurement of the length of the scribed lines was made with a precision comparator. With this instrument an accuracy of measurement of better than one thousandth of an inch was readily obtainable. This method of making deflection measurements was adopted because of the difficulty of accurate measurements of deflection under impact with any of the more usual forms of deflection-measuring equipment.

The data obtained from these measurements are shown in Figure 32 compared with the static load-deflection data obtained after the second series of impact loads were applied. This series was used because it included the 30,000 and 40,000 pound loads. It will be noted that the value of the impact pressures developed ranged from about 7,400 to about 33,000 pounds. The impact load values were those computed from the accelerometer measurements as previously described.

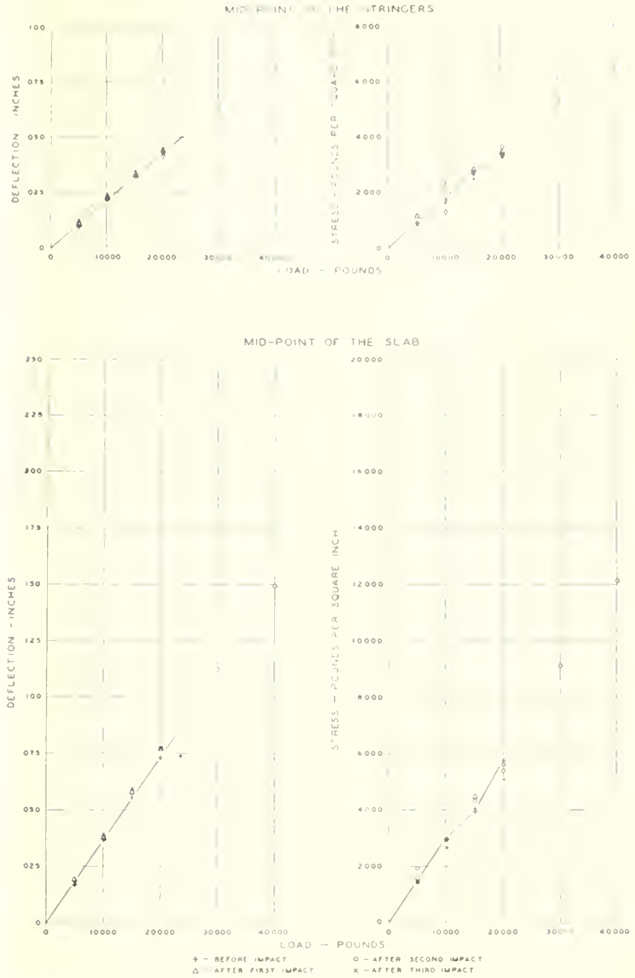


Figure 31.—Comparison of Deflections and Stresses in Second Slab Before and After Impact

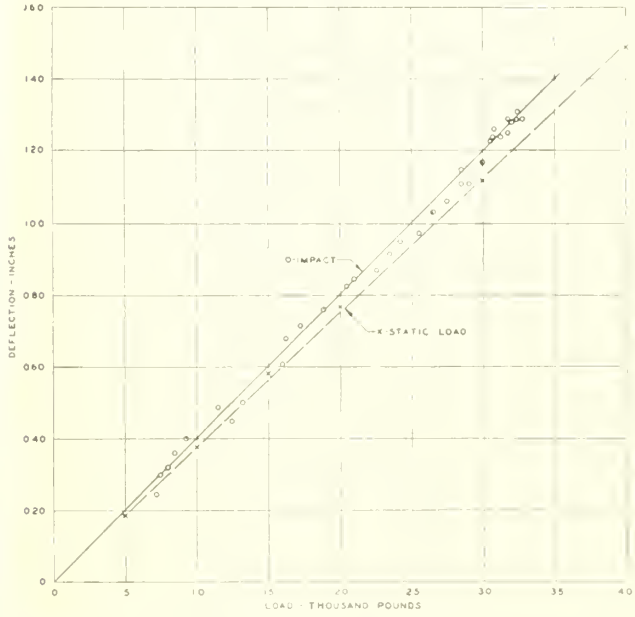


Figure 32.—Comparison of Deflections of Midpoint of Second Slab Under Impact and Static Loads

These data indicate little difference between the deflections of the mid-point of the slab under static loads and those caused by impact loads of the type imposed by the wheels of motor vehicles.

Dye test for separation of steel and concrete.—The increased flexibility of the slabs after the application of the first impact raised the question as to whether or not this was due to a breaking of the bond between the concrete and the stems of the T-bars in the vicinity of the area on which the impact was applied. A careful visual examination of this part of the first slab at the time of its demolition failed to show any evidence of such separation. After the testing of the second slab it was decided to stain the area subjected to impact with a penetrating dye so that, if any fissures had developed along the stems of the T-bars, the dye would penetrate and stain them, thus indicating their presence and extent when the slab was finally demolished. A similar area near the edge of the slab was also treated with the dye in order that the presence of fissures due to causes other than impact (as for example, shrinkage) would be revealed. The dye used was methyl violet base dissolved in alcohol and ponded over the area in question.

Some unreported experiments in the staining of fissures in concrete had shown this dye to be very satisfactory for the purpose, as it penetrates readily and produces an excellent stain.

When the slab was demolished it was found that there had been no penetration of the dye either along the sides of the T-bars or at any other point.

DISCUSSION OF THE DATA

In considering the data which have just been presented, it should be borne in mind that the deflection measurements are undoubtedly more reliable than the strain measurements because the magnitude of the displacement being measured was so much greater in the case of the deflections. For this reason personal errors, mechanical errors, and temperature errors in the measurements are generally a very small percentage of the total measurement in the deflection data. In the measurement of strains, however, such is not the case. The stresses in the steel were, in general, small and, even with the 10-inch gage length which was used, the total deformation was small, being of the general order of 0.0010 to 0.0030 inch. With such small displacements, the utmost care is necessary to keep the personal errors of strain gage manipulation and the temperature effects on the strain gage within reasonable limits.

All strain measurements were made by one operator in order to eliminate differences resulting from any personal equation. Check readings were made at periods when the air temperature agreed as nearly as possible with that which obtained during the initial readings, because it was found that, despite temperature corrections, changes of temperature of either the steel in the slab or of the gage itself could produce noticeable variations in the small strain values being measured. Corrections for temperature by means of a standard reference bar were made for all strain measurements.

It is believed that the data obtained with the strain gage in these tests indicate, with reasonable accuracy, the actual stresses which existed in the steel at the time of measurement.

Deflection of the slabs.—The general character of the deflection curves indicates that both of the slabs acted as stiff plates, giving a good distribution to the load.

In order to compare the behavior of the two slabs the following table has been prepared showing the deflections of the mid-point of the slab, of the mid-point of the stringer adjacent to the load, and of the floor beam at the stringer connections, for the 20,000-pound static load which was applied in the center of each slab both before and after the application of the first series of impact loadings.

TABLE 5.—*Gross deflections, in inches, under the 20,000-pound load applied at the center of the slab*

	Mid-point of slab	Mid-point of stringers	Floor beam at stringer connection
FIRST SLAB			
Before impact	0.087	0.046	0.014
After first impact	.113	.057	.020
Increase	.026	.011	.006
SECOND SLAB			
Before impact	0.073	0.043	0.015
After first impact	.077	.044	.016
Increase	.004	.001	.001

It is apparent that the second slab was a stiffer structure than the first slab. The deflection of the mid-point of the second slab with respect to the adjacent stringers was 0.030 inch as compared with that of 0.041 inch for the first slab. The deflection of the mid-point of the stringer with respect to its ends was 0.028 inch under the second slab and 0.032 inch under the first slab, indicating greater slab stiffness and consequent distribution of load on the stringer. These comparisons refer to deflections before impact.

The data show that the impact loadings served to increase the flexibility and thus to decrease the load-distributing ability of both slabs. In the first slab this change was very noticeable; in the second slab it was much less marked. If the deflection of the mid-point of the slab with respect to the adjacent stringers is compared it will be found that the deflection of the first slab increased from 0.041 inch to 0.056 inch (37 per cent), while for the second slab the increase was from 0.030 inch to 0.033 inch (10 per cent) after the application of the impact. Similarly, the increase in stringer deflection after the first impact was from 0.032 to 0.037 inch, or 16 per cent, for the first slab and zero for the second slab. This indicates that the impact caused a decrease in the distribution of the load on the stringers under the first slab but caused little or no change in this distribution for the second slab.

The deflection data show that after the first series of impacts subsequent loadings, either static or impact, caused little or no change in structural behavior, the action of both slabs being entirely elastic even for the 30,000 and 40,000 pound static loads.

While the causes for the differences which were observed in behavior of the two slabs have not been definitely determined, it is of interest to give some consideration to the various features of their structural design which might have been responsible for them.

The supporting structure—i. e., the walls, the floor beams, and the stringers—was the same for both slabs. As has been previously noted, the floor beams were bolted down on bearing plates and the stringers were rivet-connected to the floor beams. From these facts and such deflection data as were obtained for the floor beams and stringers, it seems improbable that any appreciable part of the differences observed was due to the substructure.

Among the structural elements of the slab proper there were several differences which may have had some influence. The base of the T-bar used in the steel mat of the second slab was appreciably thicker than that used in the first slab. (See fig. 6.) The effect of this would be to increase the stiffness of the slab and, to some extent, the stiffness of the stringers to which these members were welded. There were fewer welds between the bases of the T-bars and the stringers in the second slab than there were in the first. However, on reference to Figure 5, it will be seen that the abutting edges of the bases of the T-bars in the second slab were welded together at several places. There were no such welds in the first slab. The effect of these welds is to increase the continuity of the steel forming the bottom of the slab, and thus to increase the slab stiffness to some indeterminate extent. There is one other feature of the steel design of the slabs which probably had an important influence on the difference in stiffness noted, and that is the fact that in the second slab the upper edges of the stems of the T-bars were connected by rather heavy transverse members spaced at frequent intervals and welded in place. Since these members would be capable of receiving and carrying a considerable amount of compressive stress in the direction normal to the T-bars, their presence would give the steel mat a resistance to lateral compression quite independent of the concrete filling material.

The concrete in the two slabs was intended to be as nearly the same as possible. The proportions were the same, the aggregates were of the same quality, came from the same sources, and were of nearly the same gradations. Different cements were used for the two slabs, but the second cement was selected because its 28-day strength tests gave values almost identical with those obtained at the same age with the first cement. The first slab was placed when the air temperature was over 100° F. and the water in the mix was increased to a water-cement ratio of 0.86 before the concrete could be placed. It is probable that the concrete as it came from the mixer and was used in making the test cylinders had this water-cement ratio. It is improbable, however, that the concrete in the slab at the time the vibrating was completed had as much water in it as this water cement ratio would indicate. When the second slab was placed, the weather was much cooler and, with a water-cement ratio of 0.82, no particular difficulty was experienced in placing the concrete in the mats. Although the two sets of test cylinders cured in the damp room for 28 days showed a wide difference in concrete strength, it seems quite probable that the concrete in the two slabs at the time the load tests were made did not differ in its properties nearly as much as the strength test data from the 6 by 12 inch cylinders tested at 28 days would lead one to believe.

It is difficult to estimate just what part the concrete plays in the functioning of a slab of this type. At the time the concrete is placed it fills completely the rectangular trough between the T-bars. Subsequently, it dries out and shrinks in volume. The amount of the shrinkage which occurs in concrete during this period depends on many factors, including the kind and amount of cement and the amount of water present in the mixture. It is possible for a 3-inch prism of concrete to shrink sufficiently to draw completely away from the mold and the fact that very fine cracks were observed over the stems of the T-bars in both slabs, over the rectangular transverse members in the first slab and along the edges of the half-round transverse members in the second slab, suggested the possibility that such separation had occurred in these slabs.

These fine cracks were quite generally distributed over the entire upper surface of both slabs, were observed in many instances before any loads had been imposed, and underwent no apparent change during the application of the various loads. In a number of places in the discussion of the effect of loading reference is made to the presence or absence of structural cracks or cracks due to load and these should not be confused with the fine surface fissures due to shrinkage of the concrete which have just been mentioned.

Throughout the testing of the two slabs the behavior of the concrete was closely watched, yet no indication of a loosening of the concrete was observed at any time. When the static loads were applied at the center of the slab and the separation of the T-bars measured in the manner previously described, it was evident that even under the 5,000-pound load the measured separation at that point was sufficient to break such bond as may have existed before the static load was applied.

In spite of these evidences of separation between the concrete and the steel, it was found when the slabs were demolished that the concrete was adhering tightly to the steel; that when it was chipped off the surfaces of the steel members in contact were clean and free from rust; that the concrete showed no sign of rust stain; and, in the case of the second slab, that the dye which had been ponded on the upper surface of the slab had not penetrated the slab at any point. It must be concluded, therefore, that good bond between the steel and the concrete did exist and that this bond was maintained throughout the tests and until the time of the demolition of the slabs after an exposure of nearly a year.

In view of these facts it seems probable that the concrete acts as a filler receiving the load; that it serves to support the stems of the T-bars during bending along the transverse axis of the slab, enabling them to take full compression; that it takes some compression itself in this direction; and that during bending along the longitudinal axis of the slab it takes its proportionate share of the compression in the slab. It is indicated also that the steel surfaces which were covered by an adequate thickness of the concrete, such as the sides of the stems and the tops of the bases of the T-bars, were protected against rusting.

Stresses in the steel. The tensile stresses at the center of the lower flange of the stringers were proportional to the deflection and were very moderate for all of the loadings. In the first slab test this stress was approximately 4,000 pounds per square inch for the 20,000-pound load and about 7,000 pounds per square inch for the 40,000-pound load. In the second slab test this stress amounted to approximately 3,500 pounds per square inch for the 20,000-pound loan and about 6,000 pounds for the 40,000-pound load.

The stresses in the stems of the T-bars at the positions of maximum negative moment, over the stringers, were moderate for all of the static loads. In the first slab these stresses were of the order of 6,000 pounds per square inch for the 20,000-pound load and 11,000 pounds per square inch for the 40,000-pound load. In the second slab the stresses for these same loads were of the order of 3,500 and 7,500 pounds per square inch, respectively.

The stresses in the bases of the T-bars directly beneath the load were the highest observed at any of the points where measurements were made. In the first slab this stress was about 7,500 pounds per square inch before impact and about 10,000 pounds per square inch after impact for the 20,000-pound load. For the 40,000-pound load it was about 20,000 pounds per square inch. In the second slab the 20,000-pound load caused a stress of about 6,000 pounds per square inch, the 40,000-pound load caused a stress of about 12,000 pounds per square inch, and the effect of the impact on the magnitude of the stress at this point was very slight. If the data concerning this stress in Figure 23 are compared with those in Figure 27, it will be seen that while the magnitude of the maximum stress was not changed by the application of the impact, the distribution of the stress among the T-bars under the load was changed to some extent.

To summarize, the stress data lead to the conclusion that the 20,000-pound static load applied at the center of the slab did not cause excessive steel stresses in either of the slabs tested.

General observations.—In the construction and testing of these two slabs there were several points observed which may be of general interest.

The mats, as received, were not flat and did not lie smoothly on the stringers. It was necessary to draw these mats down as they were welded in order that each T-bar might be in bearing on the stringers.

The welding of the mats to the stringers was accomplished without difficulty with an electric welding outfit. These field-made welds were found to be sound and intact when the slabs were demolished. The welds which attached the half rounds to the stems of the T-bars in the second slab were not uniformly good, a number of defective welds being found before the concrete was placed and during the demolition of the slab.

In the construction of both of the slabs the concrete was finished off practically flush with the upper surface of the steel mats, but the finishing left a thin layer of mortar over the top of the stems of the T-bars and over the transverse members in the mats. This layer was about one-sixteenth to one-eighth inch in thickness. During the period of exposure subsequent to testing, moisture penetrated this thin layer of mortar, rusted the steel, and forced off the mortar layer. At the time the slabs were demolished, it was found that in a good many places the steel which had been exposed in this way was deeply pitted by rust.

It seems advisable that some means be developed for protecting the steel in the upper surfaces of mats which are to be used for bridge floor slabs or other exposed structures. This might be accomplished through an increase in the thickness of the concrete cover over the steel or through the application of an impermeable wearing surface to the slab. Either of these would increase the weight of the floor somewhat but would insure the durability of the structure.

Concrete for the construction of floor slabs of this type should be designed for workability as well as strength. The mix which was used in these tests was somewhat deficient in workability and would have been improved by an increase in the proportion of fine aggregate in the mixture. The protection of the steel is a very important function of the concrete and can be assured only by dense, impermeable material. Vibratory methods of placing are of great assistance in securing thorough consolidation, but a properly proportioned concrete mixture is necessary. If the concrete is not workable, the tendency will be to overvibrate in an effort to secure consolidation, and this may be carried so far as actually to cause segregation and loss of the cement paste.

CONCLUSIONS

From the data which have been presented and from observations made during the investigation the following conclusions are drawn.

1. That both slabs acted as stiff plates giving a good distribution of the load to the supporting structure.
2. That both slabs were capable of supporting static wheel loads such as are normally found in present-day traffic, without excessive stress.
3. That the effect of the severe impact which was applied during the tests was to increase the flexibility of the structure and to decrease the distribution of the load. This effect was more marked in the first slab than it was in the case of the second slab.
4. That there was nothing in the behavior of the slabs under static loads subsequent to impact to indicate that either slab had been damaged structurally by the impact.
5. That there was no loosening of the concrete by the impact.
6. That, for durability in exposed locations, the steel in the upper surface of the slabs should have more protection from moisture than was provided in the slabs tested.

BITUMINOUS CONCRETE ON CONNECTICUT AVENUE EXPERIMENTAL ROAD

The photographs on the cover of Public Roads this month depict a portion of the Connecticut Avenue experimental road which was constructed in the fall of 1912. The back cover shows the Portland cement concrete base ready for the application of the bituminous concrete surface and the appearance of the pavement one year after completion. The view on the front cover, taken at the same location in August, 1932, reveals the present condition of the surface.

This section, known as experiment No. 2 (north of Bradley Lane), is an asphaltic concrete proportioned in accordance with the District of Columbia specifications and laid 2 inches thick on a foundation of 1 : 3 : 7 gravel concrete. The experiment was divided into two sections in which limestone and trap-rock screenings, respectively, were used. Limestone dust was used as filler and the binder was a fluxed native asphalt.

The construction cost was 195.65 cents per square yard. The section has given very good service for 20 years under an increasing burden of traffic, which in 1931 reached an average daily density of more than 3,000 vehicles. Wear and depressions have developed near the gutter, and have been repaired with cold-patch mixtures of tar and stone chips. The total maintenance costs during the 20-year period have been 20.38 cents per square yard.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

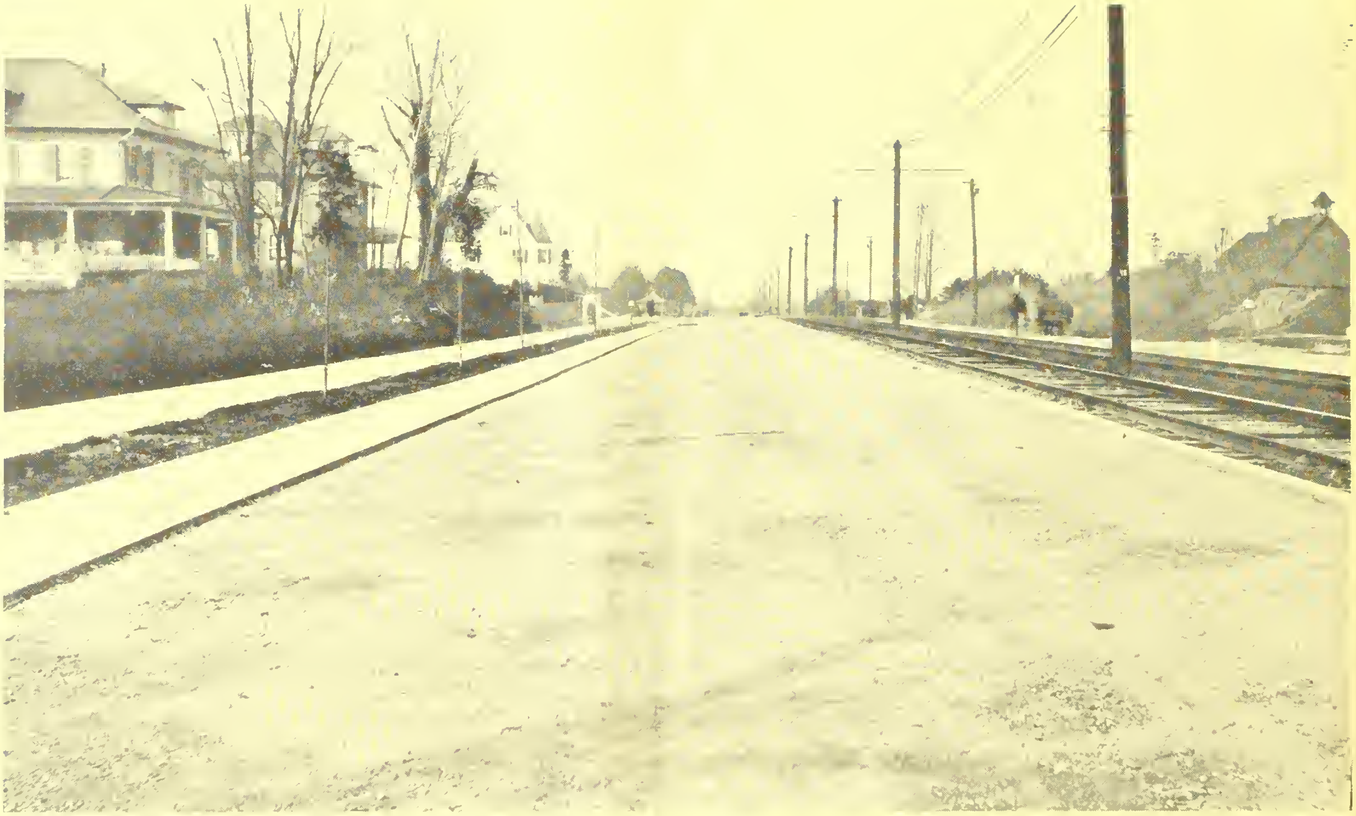
CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

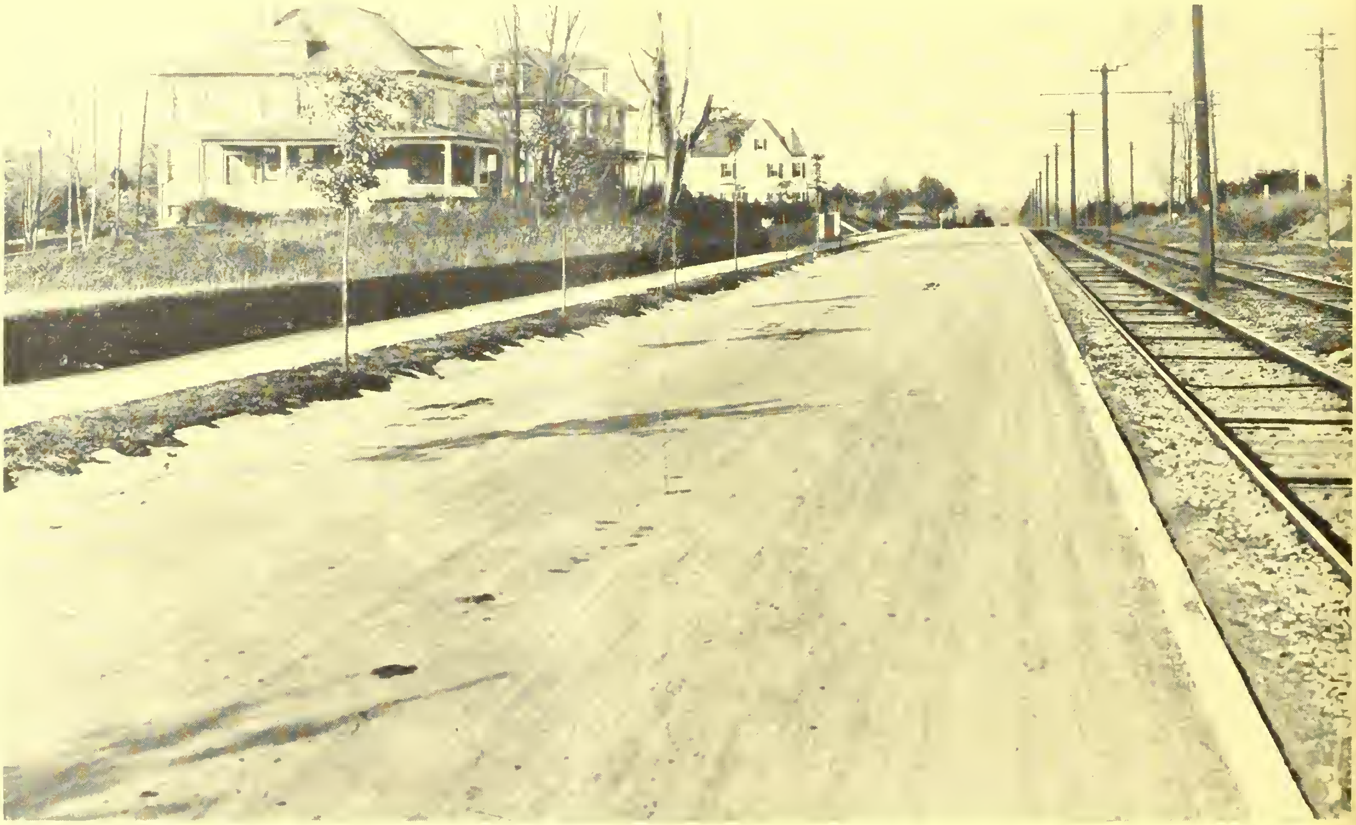
AUGUST 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
					Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$					$	$				$	
Alabama	2,405.2	1,638,442.00	797,324.95	97	62.8		62.8	58,725.70	29,362.85	0.1		0.1	5,441,650.11	Alabama
Arizona	1,263.0	2,088,285.46	1,439,484.25	74	73.3	61.7	135.0	272,862.12	185,957.62	11.4	11.2	22.6	622,993.79	Arizona
Arkansas	1,967.1	2,262,708.94	1,033,985.95	85	64.5	38.4	102.9	362,032.53	181,016.26	17.3		17.3	2,183,603.25	Arkansas
California	2,381.0	9,361,726.01	4,457,601.41	61	212.9	41.3	254.2	456,367.55	27,613.68	11.2		11.2	149,658.64	California
Colorado	1,635.7	3,788,907.63	2,005,478.03	83	151.4	45.7	197.1	110,645.42	57,288.90		12.5	12.5	1,905,004.32	Colorado
Connecticut	287.0	4,071,872.74	1,479,495.06	76	24.9		24.9	739,583.09	350,534.32	10.9		10.9	316,492.14	Connecticut
Delaware	368.7	713,766.00	356,883.00	82	22.9		22.9						167,140.82	Delaware
Florida	660.2	3,363,074.03	1,543,777.03	77	83.5		83.5	281,309.68	140,654.83	14.0		14.0	2,554,760.29	Florida
Georgia	3,196.8	5,350,186.08	2,469,490.42	69	103.7	96.6	200.3	2,358,686.34	1,050,904.76	52.4	57.1	109.5	281,185.36	Georgia
Idaho	1,321.2	2,796,422.18	1,649,205.10	83	120.9	245.9	366.8	932,602.72	443,424.06	53.1	78.9	132.0	316,199.72	Idaho
Illinois	2,747.3	24,693,217.74	11,553,396.48	81	765.0	37.3	802.3	1,269,133.53	492,831.19	32.4	5.3	37.7	1,282,514.10	Illinois
Indiana	1,878.8	9,798,463.34	4,861,495.34	73	360.3	20.3	380.6	673,152.52	336,572.23	31.9		31.9	330,586.66	Indiana
Iowa	3,321.7	5,596,084.04	2,623,065.52	55	279.8	65.8	345.6	2,192,863.61	171,393.99	92.2	32.3	124.5	17,616.03	Iowa
Kansas	3,667.4	5,011,486.05	2,442,297.86	68	246.4	112.1	358.5	1,438,881.65	696,511.49	52.8	190.3	243.1	479,029.36	Kansas
Kentucky	1,864.1	3,266,669.30	1,292,585.74	64	150.1	65.0	215.1	981,468.60	442,011.36	32.5	40.3	72.8	849,990.74	Kentucky
Louisiana	1,568.4	7,007,686.01	3,279,075.14	51	86.1	9.9	96.0						900,320.80	Louisiana
Maine	743.7	3,049,320.48	1,286,264.65	64	94.0		94.0	339,792.93	99,336.57	9.4		9.4	131,674.54	Maine
Maryland	794.5	1,756,875.28	758,362.78	37	66.3	4.7	71.0	145,106.45	51,453.72	5.4	1.5	6.9	23,131.64	Maryland
Massachusetts	821.6	6,062,373.37	2,458,736.42	84	62.7	6.6	69.3	1,092,212.22	262,592.11	11.8		11.8	242,294.74	Massachusetts
Michigan	2,119.7	8,526,956.39	3,919,188.29	73	343.5	36.8	380.3	1,189,776.40	493,277.50	32.1	38.0	70.1	1,718,969.86	Michigan
Minnesota	3,935.9	12,021,979.80	3,899,301.38	78	215.9	317.0	532.9	3,849,615.44	71,000.00	77.9	110.7	188.6	9,000.00	Minnesota
Mississippi	1,809.3	3,980,188.06	1,950,466.97	87	187.3	68.6	255.9	269,411.54	134,705.76	25.9		25.9	5,035,133.67	Mississippi
Missouri	2,935.7	6,824,609.23	2,949,242.60	68	178.0	97.1	275.1	361,486.67	173,742.54	18.6	2.9	21.5	600,971.81	Missouri
Montana	2,733.2	3,626,063.17	2,024,078.26	79	320.9	62.9	383.8	2,362,324.82	1,194,199.75	173.7	212.9	386.6	2,073,920.45	Montana
Nebraska	4,235.0	3,681,387.37	1,799,950.59	54	141.2	82.2	223.4	896,296.08	415,446.03	13.7	64.0	77.7	1,799,883.40	Nebraska
Nevada	1,305.2	1,735,515.14	1,468,486.20	65	37.1	150.0	187.1	123,288.21	109,469.42		35.0	35.0	54,712.94	Nevada
New Hampshire	424.6	551,782.15	258,211.90	45	21.0		21.0	73,997.91	29,599.16	2.8		2.8	361,439.50	New Hampshire
New Jersey	623.6	5,446,567.60	2,120,645.78	44	28.2	.5	28.7	135,582.41	33,895.60	2.3		2.3	595,474.00	New Jersey
New Mexico	2,242.7	1,860,648.39	1,296,917.91	74	84.8	25.2	110.0	437,617.69	248,140.13	11.7	28.7	40.4	963,522.93	New Mexico
New York	3,280.2	16,050,960.00	7,470,955.00	59	420.9		420.9	7,560,950.00	1,849,130.00	143.8	2.2	146.0	1,457,362.65	New York
North Carolina	2,222.3	1,198,731.16	596,538.91	82	67.9	9.0	76.9	1,003,029.10	500,697.17	114.9	17.2	132.1	3,349,069.60	North Carolina
North Dakota	5,053.5	4,050,618.86	1,941,159.65	51	456.7	383.4	840.1	1,784,693.34	703,968.66	109.2	394.8	504.0	434,922.78	North Dakota
Ohio	2,859.7	9,321,042.45	3,131,000.05	62	191.2	61.2	252.4	5,105,956.23	1,345,732.45	122.6	36.5	159.1	787,110.07	Ohio
Oklahoma	2,334.7	3,453,586.37	1,710,268.34	76	183.6	36.7	220.3	1,641,623.19	844,147.14	82.1	17.6	99.7	873,326.65	Oklahoma
Oregon	1,531.2	3,530,472.22	1,945,315.51	67	140.8	69.3	210.1	413,400.04	175,162.65	14.3	27.9	42.2	910,135.52	Oregon
Pennsylvania	3,035.0	8,671,792.49	4,210,561.60	54	283.9		283.9	4,539,861.51	1,559,801.24	134.1		134.1	658,754.45	Pennsylvania
Rhode Island	258.0	722,705.49	357,240.59	82	16.4		16.4						326,325.34	Rhode Island
South Carolina	2,049.3	2,366,835.31	1,065,169.32	99	60.4	41.4	101.8						746,947.25	South Carolina
South Dakota	4,054.7	3,894,685.47	2,115,357.51	58	285.4	188.0	473.4	998,632.24	284,520.46	76.2	93.4	169.6	379,470.73	South Dakota
Tennessee	1,690.9	1,368,046.88	680,569.31	49	41.7	21.8	63.5	923,890.77	461,945.34	36.3		36.3	2,721,634.29	Tennessee
Texas	7,765.9	14,340,432.95	6,587,011.01	70	681.6	294.4	976.0	3,813,575.10	1,734,665.44	170.5	216.5	387.0	2,404,503.19	Texas
Utah	1,137.9	1,358,969.87	990,745.08	72	84.1	112.6	196.7	585,916.45	284,143.43	46.2	25.6	71.8	523,790.08	Utah
Vermont	339.5	1,369,875.57	611,039.17	56	45.2		45.2							Vermont
Virginia	1,910.6	2,088,351.39	1,008,598.75	80	112.2	18.0	130.2	1,375,468.12	624,322.93	72.9	9.7	82.6	1,455,742.22	Virginia
Washington	1,222.0	2,977,682.18	1,417,494.35	79	99.8	13.2	113.0	772,237.61	232,300.00	17.4	6.1	23.5	743,126.17	Washington
West Virginia	894.0	1,939,105.52	819,108.45	81	47.7	10.8	58.5	1,246,166.98	473,666.56	46.6		46.6	607,091.95	West Virginia
Wisconsin	2,580.0	8,681,376.92	3,505,398.32	68	231.1	89.4	320.5	771,083.85	134,600.00	17.9	24.4	42.3	279,738.52	Wisconsin
Wyoming	1,952.3	3,413,913.88	2,127,834.84	82	179.4	215.1	394.5	174,698.46	114,631.32	22.9		22.9	18,577.01	Wyoming
Hawaii	74.8	1,442,767.53	920,136.34	43	40.7	1.7	42.4	38,476.35	38,476.35		1.9	1.9	1,335,978.89	Hawaii
TOTALS	101,504.8	242,175,216.49	112,686,297.11	69	8,260.1	3,257.6	11,517.7	56,154,483.17	19,284,846.97	2,025.4	1,795.4	3,820.8	51,425,482.97	TOTALS
Construction completed		168,307,000	77,991,000											
Balance uncompleted		73,868,000	34,695,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.



CONNECTICUT AVENUE EXPERIMENTAL ROAD, CHEVY CHASE, MD., NOVEMBER 11, 1912
PORTLAND CEMENT CONCRETE BASE READY FOR CONSTRUCTION OF BITUMINOUS CONCRETE SURFACE



CONDITION OF BITUMINOUS CONCRETE SURFACE AFTER ONE YEAR OF SERVICE, DECEMBER, 1913

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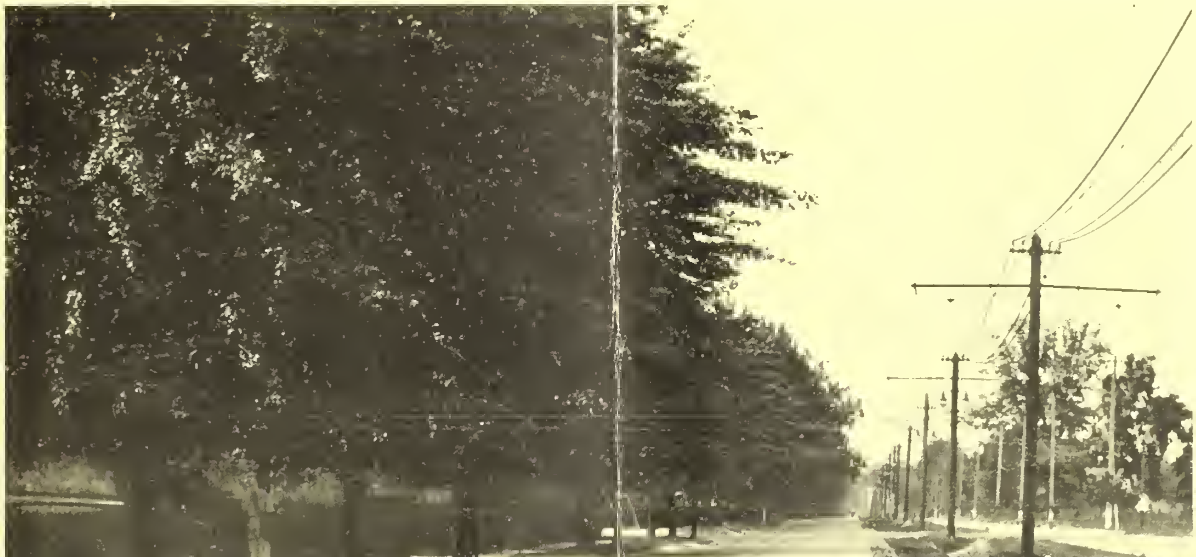
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 8 OCTOBER, 1932



A PORTLAND CEMENT CONCRETE PAVEMENT IN ITS TWENTIETH YEAR

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 8 October, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

RESEARCH ON THE ATTERBERG LIMITS OF SOILS

By ARTHUR CASAGRANDE, Research Assistant, United States Bureau of Public Roads[1]

THIS report is a detailed account of research on Atterberg's liquid and plastic limits of soils, carried on by the Massachusetts Institute of Technology, the United States Bureau of Public Roads cooperating. Special attention is paid to the development of a mechanical device for the liquid limit test, the use of which has led to a clearer understanding of the physical significance of the liquid and plastic limit tests.

The procedures described in this report are those used in this investigation. They are not intended for routine testing in subgrade work. For this purpose simpler procedures published previously in PUBLIC ROADS have been adopted by the Bureau of Public Roads. The apparatus for making the liquid limit test mechanically, however, may prove advantageous for standardizing the test procedures used in the different subgrade soil laboratories. Additional investigation of the significance of the flow curve, the flow index, and the toughness index furnished by means of the liquid limit device offers an attractive field for research in soil physics.

The most conspicuous physical property of a clay is its plasticity. This property has, therefore, been studied extensively as a basis for distinguishing between clays and nonplastic soils and for classifying different clays according to the degree of their plasticity.

The method suggested about 20 years ago by A. Atterberg (1)[2] for measuring the relative plasticity of soils has been described at length in PUBLIC ROADS (2, 3, 4). It consists of tests for disclosing the upper and lower limits of the plastic state, or the so-called liquid and plastic limits. The numerical difference between these two values is the plasticity index.

How the limit tests, in conjunction with other simplified soil tests, disclose relationships which allow an approximate determination of those physical properties which influence the behavior of subgrades, has previously been reported in PUBLIC ROADS (6).

Further and more conclusive evidence of the relationship between the results of Atterberg's limit tests and the fundamental physical property, shearing resistance, is furnished by the investigations described in this report.[3]

The more exact requirements of research in soil physics and foundation design necessitated the development of the mechanical device which eliminates almost entirely the personal equation in the determination of the liquid limit. Prior to describing this apparatus, disclosing its application in soil investigations and discussing the significance of the results furnished by its use in special investigations, it seems desirable to introduce a digest of Atterberg's paper (1).

ATTERBERG'S ORIGINAL LIMITS OF THE PLASTICITY OF SOILS

The "limits of consistency" which Atterberg established in order to get a clearer conception of the range of water contents of a soil in the plastic state are as follows:

1. The upper limit of viscous flow, at which a mixture of clay and water flows as a fluid almost like water.
2. The lower limit of viscuous flow, the "liquid limit," at which two sections of a soil cake, placed in a cup (see fig. 1) barely touch but do not flow together under the impact of several sharp blows.[4]

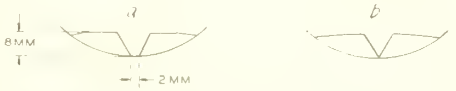

FIGURE 1.—DIAGRAM ILLUSTRATING LIQUID LIMIT TEST. FIGURE 1, *a* SHOWS WIDTH OF GROOVE AND HEIGHT OF SOIL PAT; 1, *b* SHOWS CLOSED GROOVE AFTER TEST

3. The "sticky limit," at which the clay loses its adhesive property and ceases to stick to other objects, such as the hands or a metal blade.[5]
4. The "cohesion limit" (Zusammenhaftbarkeitsgrenze), at which the grains cease to cohere to one another [5].
5. The lower limit of the plastic state, or the plastic limit, at which the soil crumbles when being rolled out into threads.
6. The lower limit of volume change or the shrinkage limit, at which further loss of moisture does not cause a loss in volume.

Atterberg considered that the amount of sand which can be added at the liquid limit without causing the soil to completely lose its plasticity is a measure of the plasticity of the soil. He found further, that the difference between the liquid and plastic limits, the plasticity index, represents a satisfactory measure of the degree of plasticity of the soil as indicated by the sand admixture.[6]

Consequently, he suggested that the difference in these limits, or the plastic range, expresses numerically the plasticity of a clay mixture and therefore may serve as a basis for classification.

PROCEDURES FOR DETERMINING THE PLASTIC AND SHRINKAGE LIMITS IN THIS INVESTIGATION

The procedure used in determining the plastic limit is essentially that previously described in PUBLIC ROADS (9). Except for the additional requirement[7] that the threads should be rolled out to a thickness of one-eighth of an inch (3 millimeters), this is Atterberg's original procedure. The procedure being used in the routine testing of subgrade soils at the Arlington, Va., laboratories for determining the shrinkage limit is described in the October 1931 issue of Public Roads (9). Except in the preparation of the soil sample, the procedure used by the writer is similar to the Arlington procedure, calculating the shrinkage limit according to the "optional methods."

---

[1] Mr. Casagrande has recently terminated his connection with the bureau and with the Massachusetts Institute of Technology and is now a lecturer at Harvard University.

[2] See bibliography at the end of this report. Numerals in parentheses refer to the articles listed in the bibliography.

[3] The significance of the Atterberg tests with respect to the ceramic industry is thoroughly discussed by Alfred B. Searle (5).

[4] There is no well-defined limit between the state of viscous flow and that of plastic flow. A valuable study on this subject is furnished by B. A. Keen and G. W. Scott Blair (7).

[5] The sticky and the cohesion limits are not discussed in this report. With respect to the former, however, it should be noted that "the sticky limit corresponds to the 'standard consistency' in the ceramic industry. Low sticky limit makes the working of the clay more difficult in agriculture, as well as in industry. The relative position of the sticky limit is increased by the addition of sand in large amounts, or organic matter in small amounts."

[6] The plasticity index, however, does not necessarily indicate the plasticity of ceramic materials as disclosed by the direct methods of measurements involving the three variables, force, shape, and time. See Searle (5), p. 272.

[7] This requirement was introduced by Professor Terzaghi (3, 8) who found that the water content at which threads will start to crumble is also dependent upon the diameter of the threads.

In the Arlington procedure the sample is molded from a paste consisting of dried soil powder mixed with water in amount slightly above the liquid limit.

In the writer's procedure the soil sample was kneaded into the shape of a disk slightly above the plastic limit, thus conforming more closely to the Atterberg definition. Dried compression test samples were used for the shrinkage limit determination.

The volume of the dried sample was determined by dividing the weight of the displaced mercury by its specific gravity.

The computation of the shrinkage limit according to the optional method as published in Public Roads (9) makes use of the formula

$$S=\left(\frac{1}{R}-\frac{1}{G}\right)\times 100$$

in which

S = shrinkage limit

R = the apparent specific gravity of the dried soil cake

$=\frac{W_o}{V_o}$, the weight of the dried soil cake divided by its volume.

G = the true specific gravity of the soil particles.

The shrinkage limit of soil fragments dried in the undisturbed state may also be calculated by means of this formula.

MECHANICAL DEVICE FOR MAKING THE LIQUID LIMIT TEST

The liquid limit device designed to perform the liquid limit test mechanically in accordance with Atterberg's definition is illustrated by the photograph, Figure 2, and the drawing, Figure 3. It consists of a brass cup and carriage and a grooving tool.

Attached to the brass cup, C, is a handle, D, so shaped that it serves first as a hook which allows the cup to turn around a pin, P, and second, as a cam follower. By means of a cam, E, turned by a crank, F, the cup is lifted to a specified height and dropped upon a polished base block, G, of hard rubber of quality which can be machined. The height to which the cup is lifted is adjusted by means of an adjustment plate, H, carrying the pin, P, and held in place by two screws, I.

This adjustment—to the height of 1 centimeter in the present instance—was the only calibration required and was made within an accuracy of ±0.2 millimeter. It was accomplished by means of a metal strip or gauge with a height of exactly 1.00 centimeter and strictly straight and parallel edges. The procedure for making this calibration was as follows.

The point where the cup came in contact with the base, which appeared as a small shiny spot on the cup after it dropped a number of times, was located. Then the adjustment plate was fixed in such a position that the height of drop was approximately 1 centimeter and the crank turned so that the cup reached its maximum height. In this position the shiny spot referred to before was readily observed, although a flash light was of appreciable assistance in this respect. The gauge was then placed with one side upon the base in such a manner that the height of its top side could be compared with the height of the shiny spot on the cup. The adjustment plate was then loosened and tightened at other positions successively until the height of the spot was equal to that of the top side of the gauge. The screws of the adjustment plate were now tightened so as to make it impossible to unscrew them by hand.

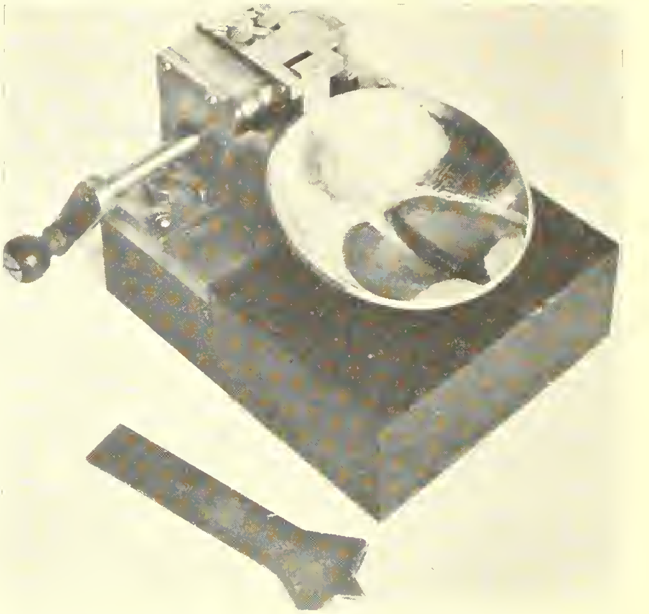

Figure 2.—Mechanical Device for Making the Liquid Limit Test

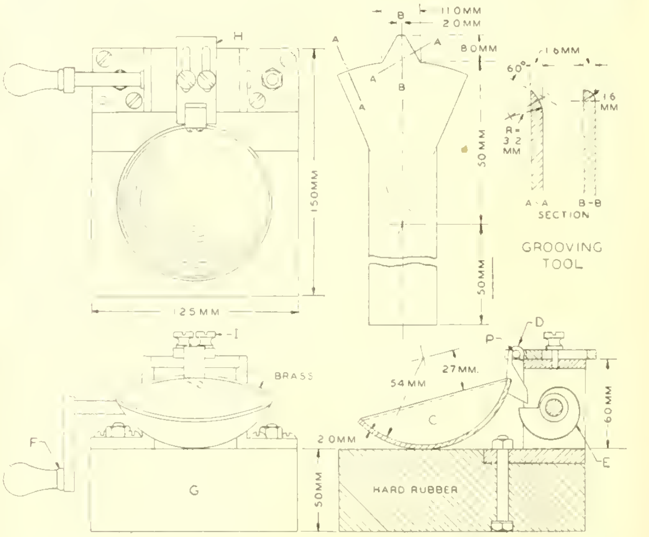

Figure 3.—Construction Details of Liquid Limit Device

In Figure 4 it can be seen that the height of drop (h_A) of point A_1 is greater than the height of drop (h_B) of the tangent to the lowest point B above the base. The difference (Δh) is dependent upon the relative position of points R, A, and C.

Had the device been designed so that no calibration were possible the whole apparatus would have required the type of very accurate workmanship furnished only by those specializing in the construction of scientific apparatus. In the design described above, however, the few dimensions (see fig. 3) specified, required only the ordinary accuracy furnished by any good workshop.

The grooving tool is made of tool steel to the dimensions shown in Figure 3 and hardened after the dimen-

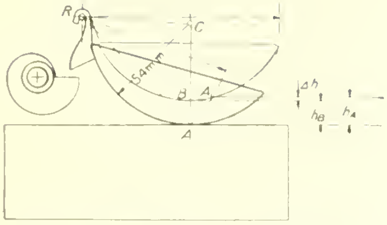

FIGURE 4.—HEIGHT ADJUSTMENT ON LIQUID LIMIT DEVICE

sions are checked. The dimensions of the groove exert an important influence on the outcome of the tests. Therefore, those dimensions of the grooving tool which control the cross-section of the groove had to be made with precision. They are the width of the top, 11.0 millimeters; the width of the bottom, 2.0 millimeters; and the height, 8.0 millimeters.[8]

THE FLOW CURVE AND ITS PHYSICAL SIGNIFICANCE

Instead of giving the liquid limit directly, tests performed with the mechanical device just described disclose the number of blows required to close the groove in soils at different moisture contents. The curve representing the relation between water content and number of blows is called the flow curve.

The flow curves are similar in character to the voids ratio-pressure curves as determined by means of confined compression tests ((8, 10) and Figures 16 and 17). They may be represented by the following equation

$$w = -F. \log N + C \text{-----------------------} (1)$$

in which

w = moisture content in per cent of weight of dry soil
F = constant, called "flow index"
N = number of blows
C = constant

According to equation 1 the semilogarithmic plots of the flow curves with the water content on the arithmetic scale and the number of blows on the logarithmic scale are straight lines. Plotting the flow curves in this manner is advantageous in several respects.

The flow curves can be drawn from a few reliable points; mistakes can be detected much more easily on the straight-line semilog than on the curved-line arithmetic plot; and finally, the flow index, F, is in this manner defined as minus the slope of the semilog plot. This equals the range in moisture content corresponding to the number of blows represented by one cycle on the logarithmic scale. Under ordinary conditions of test it is convenient to take the variation in moisture content corresponding to the cycle between 10 and 100 blows as the flow index.

Figures 5 and 6 show the flow curves of a number of soils. For the majority of soils, the points fall very close to straight lines. For some soils the points are scattered over a wider strip, although the best curves through them are straight lines. The scattering of points shown in the case of soils 7–0–2 and 49–1–1 is due to the fact that these fat clays are extremely difficult to mix into a uniform paste with their natural water content. The points of the flow curve of soil 1–93–1, dried once, are scattered on account of the very dry air in which these tests were run.

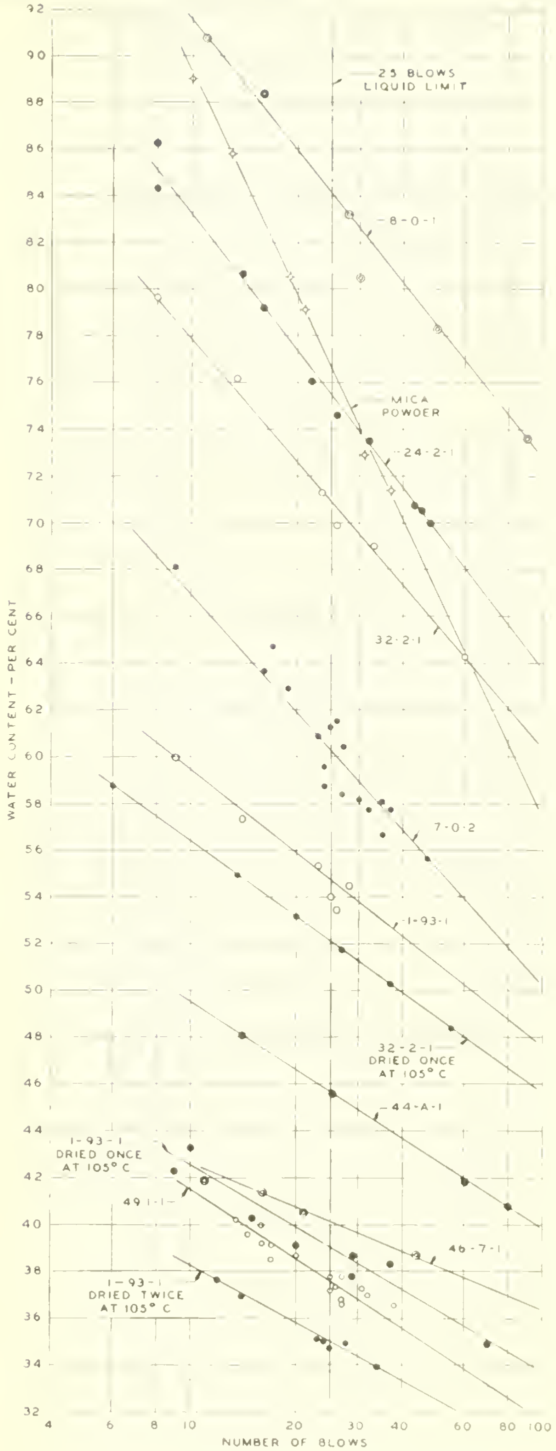

FIGURE 5.—EXAMPLES OF FLOW CURVES

In order to understand the physical significance of the flow curve and its dependent constants, one should remember that the force resisting the deformation of the sides of the groove is the shearing resistance of the soil. Hence, the number of blows required to close the groove of a soil paste represents a relative measure of the shearing resistance of this soil at this water content. It follows that the shearing resistance of all soils at the liquid limit must have a constant value (6).

This shearing resistance, as indicated by a great number of comparisons of liquid limits determined by hand by different operators, and the results of the tests which the mechanical device with a height of drop of

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[8] According to the bureau's investigation at its subgrade laboratory in Washington the tool described here and the one described in the Bureau's procedures (9) may serve equally well in tests performed on plastic soils, and the latter seems to be especially desirable for tests on the nonplastic soils.

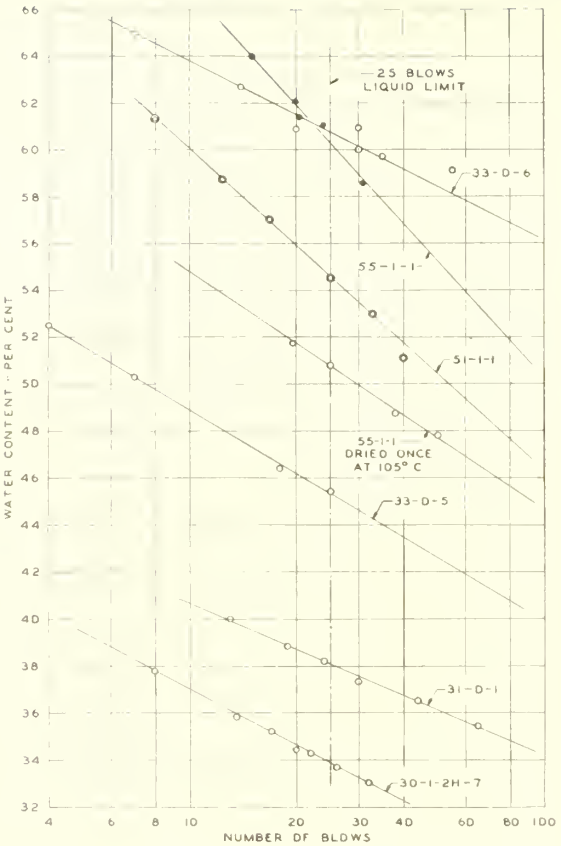

FIGURE 6.—EXAMPLES OF FLOW CURVES

one centimeter equals that possessed by the soil when requiring 25 blows to close the groove.

The shearing resistance of a given soil at the plastic limit may be many times that of the same soil at the liquid limit. There is also a wide variation in the shearing resistance of different soils at the plastic limit. This difference may be felt by hand when performing the plastic limit test on various soils. For clays this difference is commonly expressed as difference in toughness. The toughness of a clay at its plastic limit may therefore be described as the maximum stiffness or shearing resistance which it can acquire without losing its plasticity. Hence, the shearing strength at the plastic limit may be considered a measure of the toughness of a clay.

The flow index may be used as an approximate criterion for the relative magnitude of the shearing resistance of different soils at the plastic limit. In order to illustrate this relationship let us consider two soils with the same plasticity indices but different flow indices. The flow curves of these soils are shown in Figure 7. When the water content is reduced from the liquid limit by the same amount Δw, the increase (ΔN) in number of blows required to close the groove, will be smaller for the steeper flow curve.

Therefore, in Figure 7, ΔN_B is greater than ΔN_A, and the increase in number of blows is also a measure of the increase in shearing resistance. Hence, soil B will increase its shearing resistance at a faster rate when subjected to drying than soil A.

The number of blows, N, may be regarded as representing a force equal to N times the force exerted in the application of a single blow. The shearing resistance of a soil, however it may be measured, is obviously proportional to the force required to produce a given deformation (in this case the closing of the groove). It follows that N, in equation 1, may be taken as proportional to s, the shearing resistance of the soil; and we may write

$$W = -F \log s + C \text{ - - - - - - - - - - - - - } (2)$$

If s were expressed in physical units the value of the constant C would be different from that in equation 1. Otherwise the equation would be unchanged.

This relation is also correct in the range near the plastic limit according to investigations by Terzaghi and Janicsek. (See (11) p. 560.) Hence, we may assume that in the whole range between the liquid and the plastic limit the relation between shearing resistance and water content, on a semilogarithmic plot, is represented by a straight line.[9]

It should be noted that the term "flow curve," as used in this report, represents the relation between the shearing resistance (measured indirectly by means of number of blows) and the water content. In some publications on the subject of viscous and plastic flow the term "flow curve" is applied to the graphical representation of the relation between velocity (or quantity) and pressure for a fluid at a constant temperature, i. e., in the case of soil, for a paste with a constant water content.

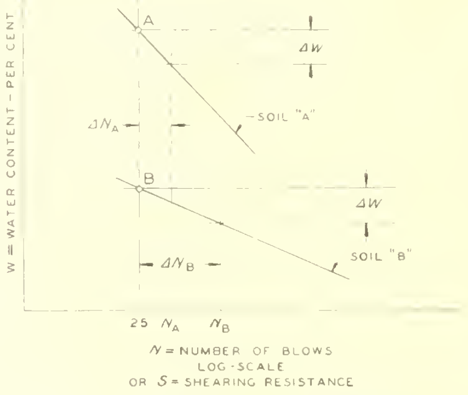

FIGURE 7.—DIAGRAM ILLUSTRATING RELATION OF FLOW CURVE AND SHEARING RESISTANCE

Assuming that the soils A and B in Figure 7 have the same plasticity indices and that this value is represented by (Δw), we draw the following conclusions:

The shearing resistance at the plastic limit of soils having the same plasticity indices is inversely proportional to the flow index. The shearing resistance at the plastic limit of soils having the same flow index but a different plasticity index is proportional to the plasticity index. Soils having the same plasticity and flow indices are, from the standpoint of shearing resistance in remolded state, alike.

In order to classify clays according to their toughness it would be necessary to determine the shearing resistance at the plastic limit by means of a direct shearing test or an unconfined compression test. An approximate measure for this quantity can be derived from equation 2 and Figure 7, using the following symbols:

L = liquid limit
P = plastic limit
$I = L - P$ = plasticity index
F = flow index
s_1 = shearing resistance corresponding to the liquid limit = constant for all soils

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[9] Investigations by B. A. Keen and G. W. Scott Blair (7) have yielded a relationship between the water content and the shearing resistance ("static rigidity") which appears similar to the flow curves of Figures 5 and 6 when plotted on arithmetic graph paper.

s_2 = shearing resistance corresponding to the plastic limit = toughness

Then T, the index of toughness at the plastic limit, is derived as follows:

$$L = -F \log s_1 + C$$
$$P = -F \log s_2 + C$$
$$L - P = I = F (\log s_2 - \log s_1).$$
$$\log \frac{s_2}{s_1} = \frac{I}{F} = T \qquad (3)$$

Instead of solving equation 3 for s_2 it is simpler to use the logarithm of the ratio between the toughness s_2 and the constant s_1 as a relative measure for the toughness which we define tentatively as toughness index T. According to equation 3 this index T is equal to the ratio of the plasticity index and the flow index.

The relative magnitude of toughness indices shown in Table 1 agrees well with the general character of these soils. Samples Nos. 7-0-2, 24-2-1, 31-0-1, 33-0-6, 44-A-1, and 55-1-1 represent tough clays. Soils which are friable at the plastic limit are the silt No. 1-93-1, when oven-dried, the kaolin No. 51-1-1, and especially the mica powder.

It may be concluded from this discussion that Atterberg's liquid and plastic limit values, supplemented by the flow index, reflect the shearing resistance of a plastic soil in remolded state at different water contents.

Table 1.—*Values obtained from Atterberg tests*

Soil No.	State of soil	Liquid limit	Flow index	Plastic limit	Plastic index	Shrinkage limit	Toughness index	Organic carbon content
		Per cent		*Per cent*	*Per cent*	*Per cent*		*Per cent*
1-93-1	Natural	54.7	11.8	32.7	22.0	28.7	1.86	2.18
1-93-1	Dried once	39.0	8.8	29.2	9.8		1.11	
1-93-1	Dried twice	34.9	8.2	27.5	7.4		0.90	
7-0-2	Natural	60.6	19.1	23.7	36.9	13.3	1.93	0
8-0-1	Natural	84.0	18.8	49.5	34.5	45.0	1.84	2.60
8-0-1	Dried once	51.0		42.0	9.0			
24-2-1	Natural	75.4	19.5	30.8	44.6	13.0	2.29	0
30-1-2H-7	Natural	33.8	8.0	21.6	12.2	17.3	1.52	0
31-0-1	Natural	38.0	6.5	21.9	16.1	20.4	2.48	0.65
32-2-1	Natural	70.8	17.5	28.3	42.5	16.7	2.43	2.00
32-2-1	Dried once	52.1	10.7	25.5	26.6		2.48	
33-0-5	Natural	45.3	8.9	28.5	16.8	30.6	1.89	
33-0-6	Natural	60.8	7.7	40.1	20.7	38.8	2.69	
44-A-1	Natural	45.6	9.6	25.2	20.4		2.12	0
46-7-1	Air dry	40.1	6.2	23.2	16.9	9.5	2.72	0
49-1-1	Natural	37.5	9.9	21.8	15.7	16.8	1.58	0
51-1-1	Air dry	54.6	13.9	33.5	21.1	29.8	1.52	0
55-1-1	Natural	60.3	16.7	26.0	34.3	21.5	2.05	0
55-1-1	Dried once	50.7	9.7	24.4	26.3	21.5	2.71	0
Ground mica powder [1]		76.6	32.0	55.6	21.0		0.77	

[1] See fig. 12.

For nonplastic soils, this relation is probably not so well defined or may even be nonexistant on account of the higher permeability of the soil which causes flow of water toward the groove under the influence of the blows. These sudden volume changes and the hydrodynamic friction (10) which replaces the actual shearing resistance of the soil obscure the physical significance of the liquid limit of nonplastic soils.

PROCEDURE FOR DETERMINING THE LIQUID LIMIT AND THE FLOW INDEX

The liquid limit was determined for soils in both their original wet state and in the dried and powdered state. In both cases a quantity of about 100 to 200 grams was thoroughly mixed and then worked up with distilled water, a spatula being used as a mixing tool, to a soil paste at approximately the liquid limit.

A small amount of this paste was then placed in the cup and smoothed off, as shown in Figure 2. The grooving tool, held perpendicularly to the point of contact, as shown in Figure 8, was drawn through the sample along the diameter through the center line of the cam follower. This was accomplished best by holding the cup in the left hand with the cam follower turned upward and away from the body. The grooving tool was held with the rounded edge toward the body and in contact with the sample. The groove was formed by drawing the tool away from the cam follower.

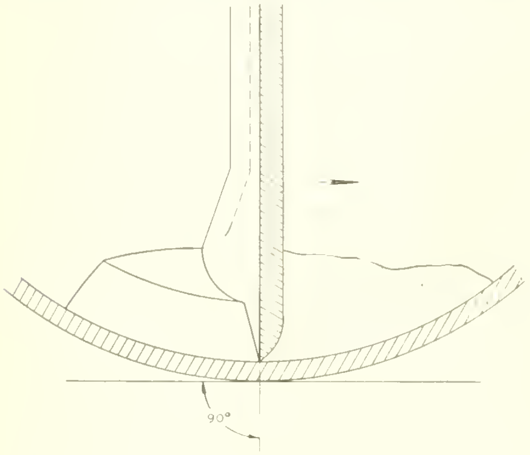

Figure 8.—Longitudinal Section Through Center of Groove, Showing Grooving Tool

With clays, a clean groove could generally be made with one stroke. Nonuniformity of the mixture, however, or sometimes the presence of coarse soil grains, tended to cause the sides of the cut to become scratched or torn locally. In such cases the sample had to be mixed and grooved again.

With some soils, especially sandy soils and soils containing organic matter, it was not possible to draw the grooving tool through the sample without tearing the sides of the groove. In such cases the groove had to be worked out gradually. When the test was run on sands it was necessary to form the groove by means of a spatula and to use the grooving tool merely for checking the dimensions of the groove.

As soon as a satisfactory groove had been made, the cup was placed on its carriage in the apparatus and the device was placed on a horizontal bed of shock absorbent material such as a layer of magazines. The purpose of this bed was to insure a constant mass, namely, that of the hard rubber block, against which the cup would strike during the test. Had the apparatus been laid on a table or clamped to it, the elastic properties of the mass upon which the cup fell would have been subject to change. Although the influence of this change upon the resulting impacts was found to be small, it was measurable and therefore, its elimination by the above means seemed desirable.

With the crank turning at the rate of about two rotations per second, the number of blows required to cause the two sides of the sample to come into contact at the bottom of the groove along a distance of about half an inch were recorded.

This operation of mixing, grooving, and recording the number of blows required to close the cut was repeated until a close agreement was observed in the number of blows required in three successive determinations. Generally, not more than three determinations were required on humid days. Because of the slow evaporation in such cases the variation in the number of blows of all three determinations on a carefully mixed sample was not larger than two blows.

Greater differences on humid days were attributed to nonuniformity of the sample. In such cases the determination of the number of required blows was repeated until the sample was mixed sufficiently to give consistent results. The average number of blows for the last three determinations was then considered as the correct value.

The effect of evaporation on dry days caused a regular and gradual increase in the number of blows required in successive determinations. In such cases the last one of at least three determinations was used, provided the values formed a fairly regular increasing series. However, there was always an uncertainty connected with the test when performed in very dry air.

As soon as the number of blows had been determined with sufficient accuracy, about 10 grams of the sample were transferred into a pair of watch glasses and weighed for moisture determination. Part of the sample around the closed groove was selected for this purpose in order to reduce the error due to nonuniformity of the sample.

The above operations were repeated at a sufficient number (at least five) of different moisture contents to furnish the data required for the flow curve. After each moisture-content determination a small quantity of the original soil mixture was put into the brass cup to replace that part removed into the watch glasses and the test performed at another moisture content.

Since the moisture content corresponding to 25 blows equals the liquid limit as performed by hand, it was deemed desirable when determining the liquid limit with the device, to use trial water contents until the consistency of the sample was such that the required number of blows ranged between 20 and 30. Until this consistency was reached no moisture content determinations were made. After it was reached but three determinations were made. Depending on whether the first sample required less than 25 or more than 25 blows, the water content of the second determination was decreased or increased. For the third test the water content was changed again in such a manner that the three points which determine the flow curve (water content against number of blows, see fig. 9) lie in the vicinity of the 25-blow ordinate. The intersection of the flow curve with this ordinate corresponds to the liquid limit.

Table 2 is selected at random from the laboratory records of the writer to serve as an example for liquid-limit determinations by means of the device. The results of these tests are shown in Figure 9.

Sample No. 30–1–2H–7 in Table 2 was tested on a very humid day. The first determination showed, nevertheless, irregularities in the number of blows.

Table 2.—*Data sheet for representative liquid limit determinations made on soil No. 30–1–2H–7 with the mechanical device on a humid day*

Number of blows, or state of soil	25, 30, 32, **32**	21, 23, **22**	24, 26, **26**	At plastic limit	
Watch glasses No	J	H	B	V	U
Weight wet, W_1, grams	38.71	36.25	36.98	30.987	25.354
Weight dry, W_2, grams	36.30	33.75	33.49	30.180	24.613
Weight glasses, W_3, grams	28.99	26.46	23.09	26.443	21.184
W_1-W_2	2.41	2.50	3.49	0.807	0.741
W_2-W_3	7.31	7.29	10.40	3.737	3.429
$w=\frac{W_1-W_2}{W_2-W_3}\times 100$	33.0	34.3	33.6	21.6	21.6

Liquid limit: L=33.7 per cent.

A detailed analysis of the causes of such irregularities and of the influence of different variables upon the accuracy of the determinations are treated later in this report.

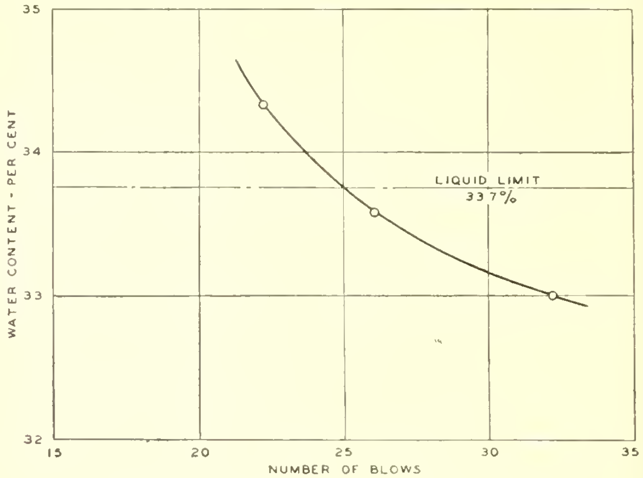

Figure 9.—Flow Curve for Soil 30-1-2H-7

In Table 2, the Figures 25, 30, 32, and 32 represent the number of blows of successive trials for the same sample (watch glasses J). The large difference between the first trial, 25, and the second trial, 30, indicates that the sample was not mixed sufficiently. The third attempt required 32 blows. The agreement between the second and the third attempt was considered sufficient. The correct value could have been taken as either the average, 31, or the last value, 32. In this case the consistency was checked a fourth time and the result, 32, was taken as representing the correct number of blows.

In the second determination of the consistency (watch glasses H) the number of blows of three successive trials was 21, 23, and 22. The last number was used for plotting the flow curve.

The third determination of the consistency (watch glasses B) gave the following three values, 24, 26, and 26.

Those numbers used for plotting the flow curve are shown in boldface type.

Table 3.—*Data sheet for representative liquid limit determinations made on soil No. 49–1–1 with the mechanical device on a dry day. Soil was dried twice*

Number of blows	18, 22, 27, 30	25, 23, 31, 35	15, 16, 17, 18	21, 22, 25, 27
Watch glasses No	E	A	G	H
Weight wet, W_1, grams	41.19	46.62	53.49	45.35
Weight dry, W_2, grams	39.02	43.71	50.49	42.57
Weight glasses, W_3, grams	33.04	35.45	42.46	35.03
W_1-W_2	2.17	2.91	3.00	2.78
W_2-W_3	5.98	8.26	8.03	7.54
$w=\frac{W_1-W_2}{W_2-W_3}\times 100$	36.3	35.2	37.4	36.8

The effect of the humidity can be seen by comparing Table 2 with Table 3. In the first case the variations in the numbers are small and chiefly due to inconsistencies in the mixture. In Table 3 each number is somewhat larger than the previous one. The first consistency determination gave the following numbers: 18, 22, 27, and 30. There is sufficient regularity in the increase of the series to warrant the assumption that 30 blows represent the consistency of the sample when placed in the watch glasses.

FACTORS INFLUENCING THE ACCURACY OF THE DETERMINATIONS

Among the factors investigated in this connection were: (*a*) Uniformity of mixture; (*b*) degree of drying the sample; (*c*) humidity of air at the time of test; (*d*)

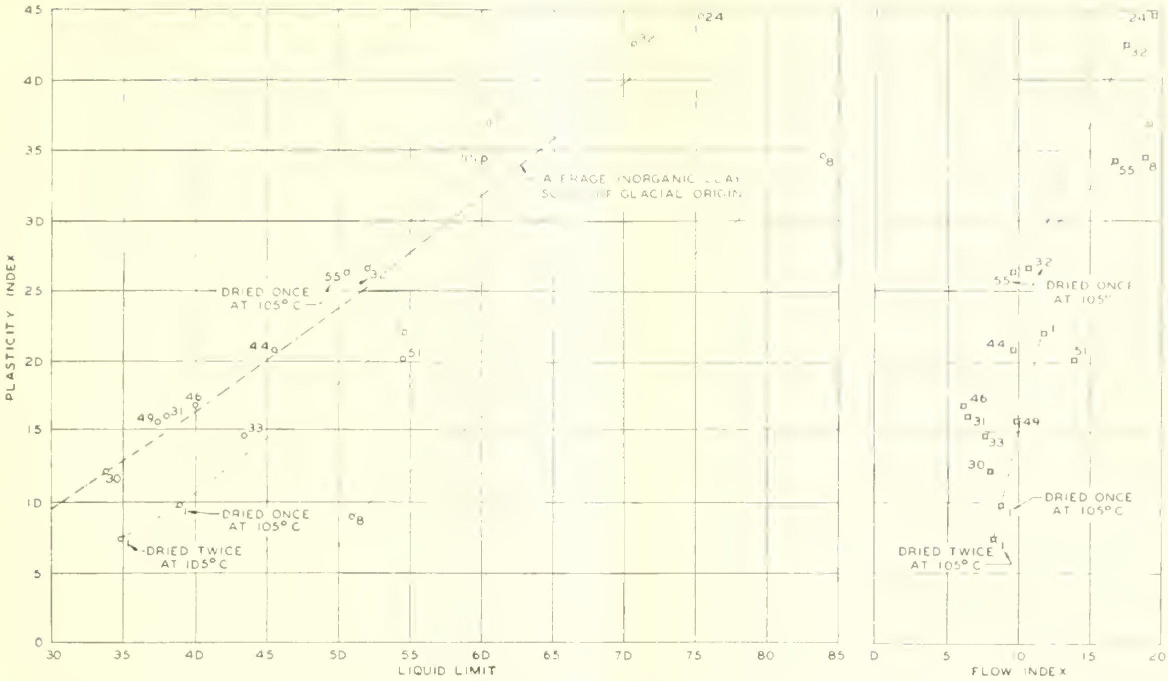

Figure 10.—Relation Between Liquid Limits and Plasticity Indices

temperature at the time of test; (*e*) accuracy of the height of drop and of dimensions of grooving tool; (*f*) weight of cup and quantity of soil used; (*g*) cleanliness of apparatus and wear on the points of contact between base and cup; (*h*) soil character.

(*a*) *Uniformity of mixture.*—It was especially difficult to obtain a uniform mixture when the test was started from the original state. Samples of very uniform clay deposits were found to consist of layers with different physical characteristics, resulting probably from variations in the formative sedimentation processes. Increase in the colloidal content of soils tends to increase the difficulty of obtaining uniform mixtures. In such cases the sides of the grooves were likely to contain streaks of soil whose true cohesion had not been entirely broken up. This condition appeared to be the cause of irregularities in the number of blows required to close the cut in successive trials, as well as a scattering of the points on the flow curve, as shown in the case of soil 7–0–2 (fig. 5) and soil 49–1–1 (fig. 6).

(*b*) *Degree of drying the sample.*—Flow curves determined for soils in the dried and powdered state will give very consistent results because the powder can be mixed more thoroughly than the natural wet soil. Drying soils, however, may cause a slight drop in the liquid and plastic limits. Oven drying of some soils may cause a very appreciable drop in the limits and for this reason soils are air dried in the routine subgrade tests.

This drop is especially important in the case of soils containing organic matter. Their liquid and plastic limits change somewhat when exposed to air and considerably when they are oven dried. In Table 1 examples of this kind are represented by an organic clay soil from Turkey (No. 32–2–1), an organic silt from Cambridge, Mass. (No. 1–93–1), and an organic clay from New London, Conn. (No. 8–0–1).

The drop of plastic and liquid limit due to drying is represented graphically in Figure 10. It will be noticed that the drop occurs along approximately parallel lines. The trend of these lines is the same as that of the very flat curve representing the relation between liquid limit and plasticity index for average inorganic clay soils of glacial origin, shown in Figure 10 as a dot-dash curve.

The plastic and liquid limits of most inorganic clay soils display slight decreases after drying. However, this drop is very small, and it was found that by saturating the soil for several days the original plasticity could often be restored. The only inorganic clay soil for which the plasticity decreased an appreciable amount as a result of oven-drying is soil No. 55–1–1, a marine clay deposit from the St. Lawrence River Valley, Quebec. From the graphical representation in Figure 10 it can be seen that this decrease is only about one-half of the decrease observed for the three soils with organic content.

Apparently the cause of this phenomenon is the baking together of the grains of colloidal size. Most inorganic colloids seem to be reversible, while organic colloids are partially destroyed even at temperatures of 105°C. and therefore must be considered as irreversible.

(*c*) *Humidity of air at time of test.*—On very dry days the evaporation of moisture from the sample proceeded fast enough to make subsequent determinations of the number of blows appear as a rapidly progressing series of numbers. Therefore, in order to prevent additional irregularities in this series on account of differences in water content, the samples had to be mixed quickly and thoroughly after each trial. In addition the mixture which was used for test was protected from evaporation during the test and, furthermore, the transfer from the tested soil cake to the watch glasses was carried out as quickly as possible.

(*d*) *Temperature at time of test.*—A series of tests by Atterberg (Table 4), which were carried out at 7° and 24° C., show practically no effect of the temperature. A few careful tests which the writer performed at temperatures between 35° and 40° C. showed no difference from the values obtained at 20° C.

(*e*) *Accuracy of height of drop and dimensions of grooving tool.*—As would be expected, it was found that the effect of the blows varies approximately with the square of the height of drop. Results of the tests are

Table 4.—*Effect of temperature on the liquid and plastic limit determinations*

Soil	Temperature	Liquid limit	Plastic limit
	Degrees C	*Per cent*	*Per cent*
1	7	31.5	25.7
	24	30.5	26.0
2	7	41.6	25.0
	24	40.5	23.8
3	7	24.5	18.9
	24	24.4	17.6
4	7	58.8	52.8
	24	58.9	53.9

shown in Figure 11. An error of 0.1 millimeter in adjustment of the height was found to correspond with a change of 2 per cent in the number of blows. Adjusting the height of drop within an accuracy of ±0.2 millimeter proved satisfactory for ordinary purposes.

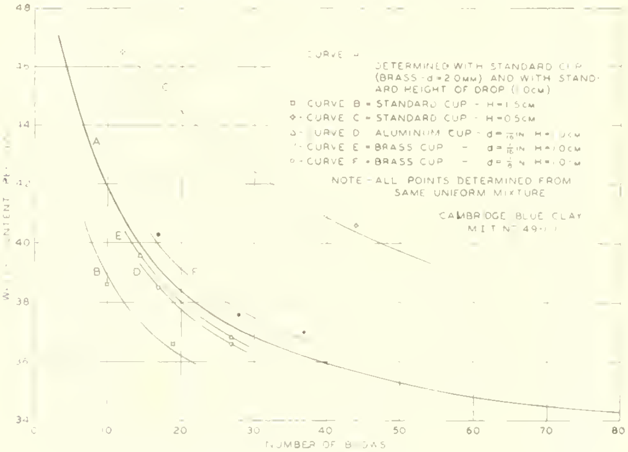

Figure 11.—Influence of Weight of Cup and Height of Drop upon Flow Curves

Test results indicate that that dimension of the grooving tool which determines the width of the bottom of the groove should be accurate within ±0.05 millimeter.

(*f*) *Weight of cup and quantity of soil used.*—Figure 11 also gives the results of tests to determine the effect of weight of cup. The results show that the number of blows required to close the cut increases slightly with the weight of the cup. The weight of the standard cup, including the cam follower, is 200 grams, and should be adhered to within 10 grams.

As expected, the tests show that the quantity of soil has no appreciable influence on the test results. The quantity should be large enough to permit the cutting of a well-shaped groove.

(*g*) *Cleanness of apparatus and wear on the points of contact between base and cup.*—A drop of water or small quantity of soil, located at the point of contact between the cup and base, will reduce the effect of impact to a considerable degree and cause appreciable errors. Wear on the points of contact between base and cup have not been found to produce appreciable error.

(*h*) *Character of soil.*—Difficulties were encountered in determining the flow curves of nonplastic soils, largely because of insufficient adhesion to the cup and excessive permeability, the latter property causing excessive water content in the region of the groove. Preliminary tests indicate that the principle of the flow table, as used in cement and concrete testing, would provide a more suitable apparatus for the determination of the flow curves of nonplastic soils than the liquid-limit device in its present form.

CONSIDERATIONS IN THE PRACTICAL USE OF THE CONSISTENCY LIMITS

In order to understand thoroughly the practical use of the limit tests, one should have some conception of (*a*) the effect of grain size and shape of soil grains and (*b*) the effect of the structure on the plastic properties of the soil.

(*a*) *Effect of size and shape of grains.*—The hypothesis that plasticity and other important properties of clay are caused by the presence of scalelike particles was first advanced by Voigt in 1897. Later, and independently, other scientists arrived at the same conclusion. Reliable evidence for the truth of this assumption was, for the first time, presented by A. Atterberg (12).

Professor Terzaghi (10 and 11) observed that clays have a much lower permeability than sands with the same equivalent grain diameter, and concluded that the volume of a clay particle must be much smaller than that of a bulky grain of the same size. This indicates that clay particles are of scalelike shape.

Professor Goldschmidt (13) succeeded in finding a satisfactory explanation for the action of scalelike particles in soil. He showed that the surfaces of scalelike particles are charged with electricity wich affects the arrangement of the bipolar molecules in a liquid like water. When the clay powder is mixed with a liquid consisting of molecules which are not distinctly bipolar (e. g. carbon tetrachloride), the mixture does not acquire plasticity. The percentage of scalelike particles in common clays varies between 10 and 30 per cent, of which only that fraction smaller than about 0.2 micron (equivalent grain diameter) may be considered effective from the standpoint of plasticity.

From the above information it follows that in most cases the grain-size distribution of a soil does not yield any information on plasticity. As a striking example, let us compare the grain-size curves of a plastic clay, quartz powder,[10] feldspar powder,[10] and mica powder,[10] shown in Figure 12.

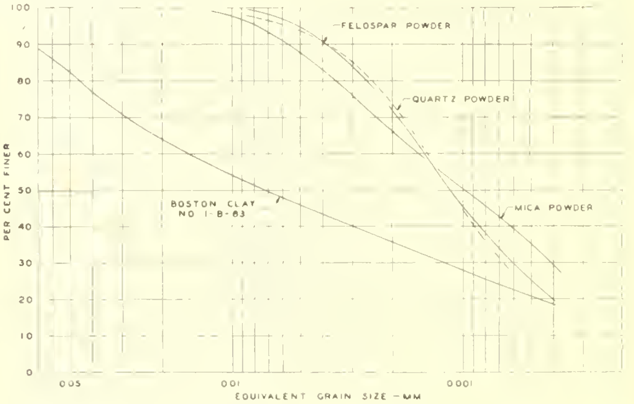

Figure 12.—Grain Size Curves of Quartz, Feldspar, and Mica Powders, Compared with a Plastic Clay

The ground quartz and feldspar consist of bulky grains and have no plasticity. The ground mica is slightly plastic. Table 5 contains a comparison of the plasticity of these four materials with the percentage of grains smaller than 5 microns (clay according to the U. S. Bureau of Chemistry and Soils), smaller than 2 microns (clay according to the international classification), and smaller than 0.2 micron. The information contained in Figure 12 and in Table 5 was furnished by Dr. L. Jurgenson.[11]

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[10] Ground wet in a ball mill.

[11] Research associate, Massachusetts Institute of Technology.

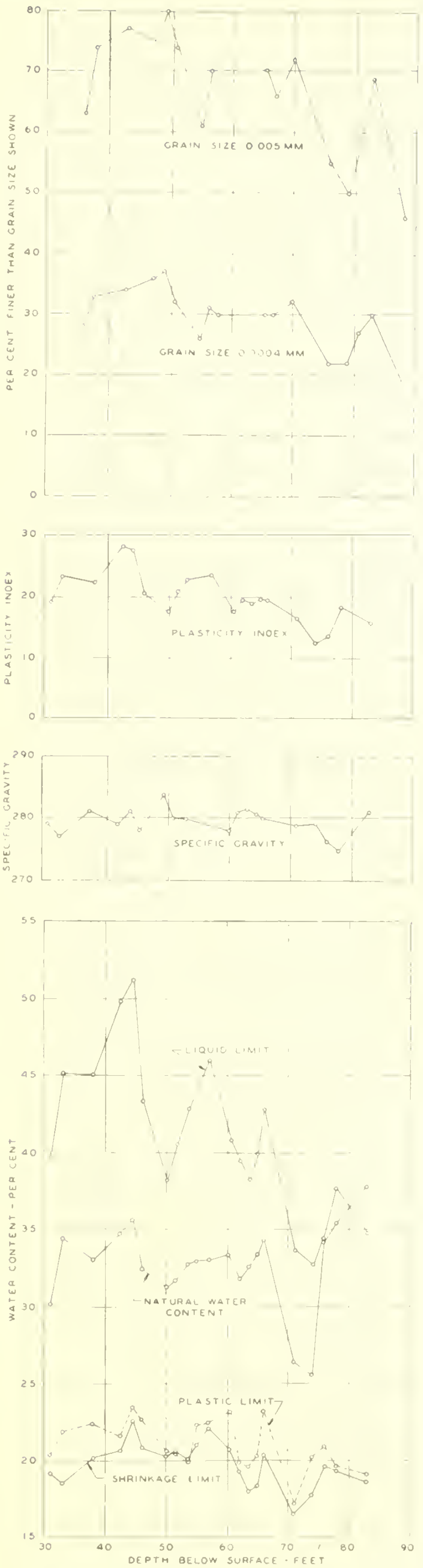

Figure 13.—Comparison of Variation of Limit Values and Grain Size in Soil Layers at Different Depths

Table 5.—*Comparison of grain size and plasticity for different materials*

Material	Liquid limit	Plasticity index	Less than 5 microns	Less than 2 microns	Less than 0.2 micron (estimated)
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Boston clay, No. 1-B 83	35	16	46	36	12
Mica powder	76.6	21	88	66	12
Quartz powder	No plasticity		94	74	4
Feldspar power			95	72	8

Different soil layers from the same deposit are often related in character so that changes in grain-size distribution may reflect a corresponding change in plasticity. An example is shown in Figure 13.

(*b*) *The effect of the structure of clay.*—The mechanical properties of clay below the zone of weathering and in undisturbed state are different from the properties of a sample which is thoroughly remolded. A detailed discussion of the structure of undisturbed clay is beyond the scope of this report. The reader is referred to (15) and (16). As an example for the effect which remolding has on the physical properties of clay, the pressure-void ratio diagrams for an undisturbed clay and the same clay remolded, are shown in Figure 14.

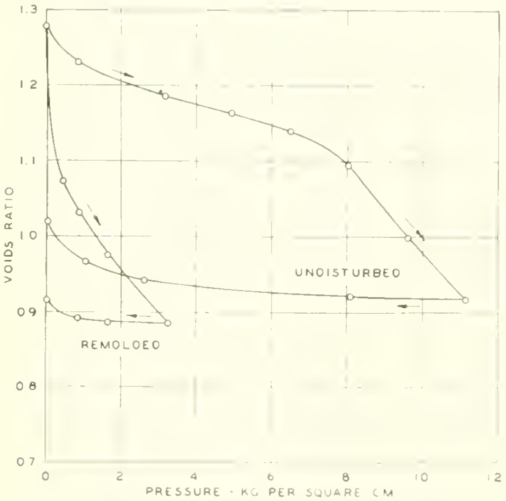

Figure 14.—Relation Between Pressure and Voids Ratio in Undisturbed and Remolded States, for Laurentian Clay, No. 56 103

Since the liquid and plastic limit tests can be performed only after the natural structure of the clay is destroyed by remolding, they are incapable of reflecting any mechanical properties of the clay in undisturbed state. Consequently, at the liquid limit a clay deposit may have very appreciable strength, whereas the shear resistance of the same soil at the liquid limit, remolded, will be very small. Some Laurentian clays may be so rigid at the liquid limit that they can carry a building load of 5 tons per square foot without excessive settlement.

In contrast to the liquid and the plastic limits it is interesting to note that the shrinkage limit can be determined in both the undisturbed and the remolded state (9). The results may differ in varying amounts depending upon the structure of the soil. The difference in shrinkage limits when determined from both the undisturbed and remolded state may be especially useful in estimating rapidly the structural characteristics of the material. Table 6 illustrates the difference observed between shrinkage limits of a soil in the remolded and undisturbed state and also shows that the plastic limit may be lower than the shrinkage limit in the undisturbed state.

Table 6.—*Tests on samples in both the disturbed and the undisturbed states*

Type of clay	Laurentian clays		Boston clay
Sample number	55-1-1	56-D-1	49-1-SP
	Per cent	*Per cent*	*Per cent*
Water content in natural state	56.2	38.0	30.6
Plastic limit from natural wet state	26.0	23.0	21.5
Shrinkage limit undisturbed	34.7	28.6	20.3
Shrinkage limit remolded	21.5	20.7	16.3

SELECTION OF SAMPLES FOR FINAL TESTS ILLUSTRATED

Glacial and river deposits are often nonuniform, varying from coarse sands and gravels to plastic clays. In such cases a great number of soil samples must be taken from borings or test pits in order to investigate these deposits for foundation purposes. For a rapid classification of these samples, to facilitate the selection of a few representative ones for more elaborate tests (compressibility, shearing resistance, etc.) (14) Atterberg's liquid and plastic limit tests prove very satisfactory.

The distribution of nonplastic silt and sand layers is of utmost importance for computing the rate of settlement. Even very fine layers of more permeable material will act as drains for water which is pressed out of the compressible clay layers during the process of consolidation.

The ability of the limit tests to disclose differences in soil, visibly the same, is illustrated by Figure 13, referred to previously. These soil samples of Boston clay were taken at different elevations in the same drill hole in an investigation of building settlement carried on under the supervision of Prof. Glennon Gilboy. In spite of the fact that the samples showed the clay deposit to be uniform in color and appearance and without any stratification, the limit tests disclosed important differences in the different samples.

In this manner they facilitated the selection of sample for the determination of the consolidation characteristics by means of confined compression tests (8).

SUMMARY

The foregoing investigation indicates that reliable results are to be expected from the use of the flow curve apparatus under the following conditions:

(1) That the device be calibrated to an accuracy of 0.1 millimeter for a height of drop of one centimeter.

(2) That the device be placed in a horizontal position and be cushioned by relatively soft material, such as newspapers or magazines.

(3) That the surface of the base and the outside of the cup be wiped free of moisture or soil powder before the number of blows are determined.

(4) That the sample be carefully mixed before the test is performed, especially when it is started with soil in the natural wet state.

(5) That a straight groove be made by holding the cup in one hand, cam follower turned upward and away from the body, and drawing the standardized grooving tool toward the body along a line passing though the center line of the cam follower and the point of contact between cup and base in the manner previously described. (Fig. 8.)

(6) That the crank be turned at the rate of two rotations per second until the bottom of the cut is closed for a distance of about one-half inch.

(7) That the operations be repeated until the last three numbers of blows are in substantial agreement.

(8) That at least two or three determinations be made at different water contents in the vicinity of 25 blows for the determination of the liquid limit, and at least five determinations, ranging between 10 and 40 blows be made for the determination of the flow curve and the flow index.

Irregularities in successive determinations of the number of blows were found to be due to—

1. Irregular shape of groove, side walls scratched due to inconsistencies of sample or because the edge of the grooving tool was not cleaned.
2. Nonuniform mixture of sample in natural state or rapid evaporation and insufficient mixing after every determination.
3. Grooving tool held at different angles instead of perpendicular to the tangent at the point of contact.
4. Moisture or dry soil powder on the points of contact of base and cup.

Generally, with respect to the plasticity tests it was found that—

(1) The flow curve on a semilogarithmic plot is a straight line.

(2) The liquid limit is the water content corresponding to 25 blows as determined by means of a standardized device and procedure.

(3) The flow index is a constant indicating the slope of the flow curve when plotted on semilogarithmic paper. It is equal to the difference of the water contents corresponding to 10 and to 100 blows.

(4) The liquid and plastic limits of soils determined with their natural water content are for inorganic clay soils equal or slightly higher than the limits determined from oven-dried state. For soils containing organic matter, the difference may be very large.

(5) The change in plasticity caused by drying is accompanied by a change in other physical properties, especially the cohesion and the permeability of the soil.

(6) The liquid limit is the water content at which the raw material of a soil in remolded state possesses a very small but constant shearing resistance.

(7) The shearing resistance at the plastic limit may be different for different soils. The flow index may be used as a relative measure of this difference.

(8) For soils having the same plasticity indices, the shearing resistance in remolded state at the plastic limit is inversely proportional to the flow index.

(9) For soils having the same flow indices, the shearing resistance in remolded state at the plastic limit is proportional to the plasticity index.

(10) Soils having the same plasticity and flow indices, have the same shearing resistance in their remolded state at corresponding intermediate states.

(11) The difference between the shrinkage limit in undisturbed and remolded state is caused by the character of the structure in natural state, and therefore, may be used as a measure of the amount of disturbance resulting from thorough molding.

(12) The physical significance of the liquid limit of a nonplastic soil is fundamentally different from that of a plastic soil. It appears that this test when applied to slightly cohesive or cohesionless soils has more the character of a stability test.

(13) The plastic and liquid limit, and the flow index yield qualitative information on the shearing resistance of the soil in remolded state.

(Continued on p. 136)

THE NEW HAMPSHIRE TRAFFIC SURVEY

Reported by L. E. PEABODY ,Senior Highway Economist, United States Bureau of Public Roads

DURING the summer of 1931 the State Highway Commission of New Hampshire cooperated with the Bureau of Public Roads in a survey of highway traffic upon the trunk-line, State-aid, and town road systems of New Hampshire. Field observations were taken at the same locations and with the identical schedule of operation as in an earlier survey [1] in 1926. Figure 1 shows the locations of stations at which field operations were carried on in 1926 and 1931, and a graphical comparison of 1931 traffic with the 1926 forecast.

When account is taken of the unpredictable shifts in highway traffic, the data obtained in this survey afford a reasonably close check upon the results of the 1926 survey and the traffic forecast made at that time.

Traffic observed in 1931 upon the 383 routes or sections of routes aggregated 470,992 vehicles per day, as compared with a forecast of 408,549 vehicles per day made five years earlier. Thus the traffic in 1931 was greater by 15.3 per cent than the amount forecast in 1926. This is the gross comparison, and there is wider divergence when the comparison is made for individual sections of routes.

Table 1.—*Comparison of 1926 traffic forecast with actual traffic in 1931*

Routes with average daily traffic—	Total motor vehicles		Ratio, 1931 to 1926
	Forecast, 1926	Actual, 1931	
Less than 200	8, 940	12, 953	1. 449
200 to 499	23, 390	39, 720	1. 698
500 to 999	56, 940	75, 639	1. 328
1,000 to 1,499	78, 550	95, 202	1. 212
1,500 or more	240, 729	247, 478	1. 028
	408, 549	470, 992	1. 153

In Table 1 the forecast and actual traffic are compared by specific classes of routes; that is, those with less than 200 vehicles per day, 200 to 499 vehicles per day, etc. This tabulation shows that the actual traffic was relatively higher than the forecast upon routes carrying less than 500 vehicles per day in comparison with the more heavily traveled routes. The aggregate traffic on all routes carrying 500 or more vehicles per day exceeds the forecast by less than 9 per cent. Upon the most important routes (those carrying 1,500 or more vehicles per day) the actual traffic exceeded the forecast by less than 3 per cent.

If the traffic of 1931 is compared with the traffic classification of routes [2] made in 1926, it will be found that the traffic classification, based on 1926 traffic and a forecast based on that traffic, is closely accurate. In general, the variation from the forecast is well within the ranges of the traffic classifications set up in the earlier report. These ranges of traffic density, rather than the individual figures, are the significant factors in the study of a highway system; and the differentials between the 1926 forecast and the 1931 traffic are insufficient to change appreciably the traffic classification of these routes as originally set up.

While the relative variations are greatest upon the light traffic routes (Table 1) especially those carrying under 500 vehicles per day, variations on these routes represent relatively small numerical differences in traffic count. This point may be brought out in still another way: On more than half the 383 routes or sections of routes for which traffic was forecast, the actual traffic in 1931 checked within 200 vehicles per day, and on 82 per cent of all routes, the differences were less than 500 vehicles per day.

If the comparison be made by specific routes, much the same conclusions are reached, as is shown in Table 2. The routes in this table are arranged in order of magnitude of the 1931 traffic. Upon the heavily traveled routes, the 1931 traffic is slightly below that forecast and, as the traffic decreases, the forecast traffic becomes greater in relation to the observed traffic.

Table 2.—*Comparison of 1926 traffic forecast with actual traffic in 1931 by specific routes*

Route	Location	Average daily density of 1931 traffic	Differences from 1926 forecast
			Per cent
U. S. 1	From Massachusetts line via Seabrook, Hampton, and Portsmouth to Maine line.	8, 078	−8. 5
U. S. 3	Lakeport via Franklin, Concord, Manchester, and Nashua to Massachusetts line.	4, 577	−12. 4
101 and 101A	Keene via Peterboro and Milford to Nashua	1, 954	+12. 2
18	From Vermont line via Littleton, Twin Mountain, Glen, and Conway to Maine line.	1, 704	+10. 2
U. S. 3	Colebrook via Lancaster, Whitefield, and Twin Mountain to Plymouth.	1, 532	+19. 9
16 and 28	Conway via Ossipee and Milton to Rochester	1, 428	+20. 0

The greater increase in traffic upon the more lightly traveled routes is not definitely a result of congestion upon the heavy-traffic routes, but is probably due to improvement, in surface, alignment, and grade, of more direct routes and of the routes carrying the lighter traffic, as well as to the desire of motorists to avoid heavier traffic. In general, these routes of lighter traffic are less highly improved; but, despite the poorer conditions with respect to grade, alignment, etc., it may be possible to travel as comfortably and as rapidly upon them as upon the more congested routes. In many cases, new scenery is a factor in the choice.

The improvement in the light-traffic routes since 1926 has been extensive. For instance, travel on route 10 between Hanover and Woodsville now averages more than 1,500 vehicles daily. Route 111 from Wentworth to Orford, joining route 10 at Orford, has a traffic average of more than 480 vehicles per day, or three times the amount forecast for this section in 1926. The increase upon this relatively lightly traveled route is due wholly to the completion of route 111 with a surfaced road since the survey of 1926, as this trunk line was not improved at that time.

Many of the divergences between the 1931 forecast of the 1926 survey and the actual count in 1931 are due to the completion of existing trunk line layouts created by the legislature, upon which little or no construction was in existence at the time of the 1926 survey. The trunk line completion act of the 1929 legislature provided for the completion of all routes designated as trunk lines by the close of the 1930 construction season.

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[1] Report of a Survey of Transportation on the State Highways of New Hampshire (1927). A number of stations which were not operated at their 1926 locations, and certain 1926 stations which were not occupied in 1931, are not included in the comparisons here given.

[2] Report of a Survey of Transportation on the State Highways of New Hampshire, 1927, p. 57 et seq.

Construction activities of this period resulted in several differentials, of which the following are examples. At station 65 in Somersworth traffic on route 16 southeast to Somersworth was 2,256 vehicles per day less than the forecast, while traffic on route 16–A south to Dover was 2,489 vehicles per day more than the forecast. Traffic north on route 16 at this station was 3,300 vehicles per day or very close to the 3,230 vehicles per day forecast in 1926 for this route. The shift in southbound traffic from route 16 to route 16–A is clearly a result of the complete construction with modern pavement of route 16–A, a much shorter and better route for through traffic. The completion of route 16–A places it in the major traffic classification, for practically all but local traffic now utilizes this shorter direct route in preference to the more circuitous route 16, which has now become of secondary importance.

At station 99 traffic toward Exeter was within 91 vehicles per day of that forecast. Traffic on route 101–C was 241 vehicles per day less than the forecast traffic, while traffic on the shorter State-aid route to Hampton Falls was 151 vehicles more than forecast. The improvement of the shorter State-aid route has drawn traffic to it somewhat in preference to route 101–C. In addition this diversion indicates the general tendency for traffic to seek out shorter, more direct routes in good condition, in preference to main routes on which there is congestion, such as route U. S. 1, the adjoining heavily traveled highway.

Construction activities may show their effects upon the routing of traffic at a considerable distance from the location of the improvement and thus affect the accuracy of a forecast. For example, at station 72, several miles north of Berlin on route 16, highway construction projects were under way during the period of 1931 field work. Traffic on route 16 was 87 vehicles per day less than forecast, while the traffic on route 110 northwest of Berlin exceeded the estimate by 93 vehicles per day.

Again, traffic on U. S. 3 and U. S. 4 between Concord and Laconia via Franklin was much reduced from that estimated in 1926, because of construction on these sections, while the traffic on route 106 between Concord and Laconia was considerably in excess of the amount forecast. In this latter case the increase on route 106 is evidently due in great part to the completion of the original construction on this shorter and more direct route from Concord to Laconia through less populated territory, avoiding the city of Franklin and the larger town of Tilton with their accompanying traffic delays. The construction on route U. S. 3 in Franklin and Tilton was avoided to a great extent by the use of the State-aid route from Boscawen through Northfield to Tilton, upon which there was a traffic increase of approximately 50 per cent in excess of that forecast, because of the use of this optional route during the construction on U. S. 3. While a slight increase in traffic on route 106 may be attributed to construction on U. S. 3, check surveys operated by the State highway department between the 1926 and 1931 cooperative surveys have indicated a substantial increase in traffic on route 106 above a normal expected increase, which may be attributed wholly to the complete construction of this route, which under average conditions shortens the driving time between Concord and Laconia by from 15 to 30 minutes.

Although no construction took place at station 64 during the period of the survey, there is evidence of considerable change in traffic routed through this station. Traffic between Claremont and this station during 1931 was 1,742 daily vehicles, or very close to the forecast of 1,730 vehicles per day. Traffic west through Vermont on route 12 is 385 vehicles per day greater than the forecast, while the traffic north on the township road to Plainfield is 292 per day under the forecast.

Construction activities which permit the use of the highway produce less effect upon traffic movement on the lightly traveled routes. As an example, traffic on all routes leading from station 4 is in excess of that forecast, although highway construction during the period of the survey was noted at this station. The excess north on the township road to Lancaster was 32 vehicles, west on U. S. 2 to Riverton, 91 vehicles, and east on U. S. 2 to Jefferson, 65 vehicles per day.

Improvement of the Concord-Keene highway was completed in 1930. Although portions of this route were improved in 1926, there were many sections of the route awaiting improvement. The Wolfeboro-Alton section of route 28 was completed in 1930. Route 11 north of Alton to the junction of route 28 was heavily used by traffic during 1931 to avoid construction on U. S. 3. This section has been completed since 1926. On all of these sections the traffic use during 1931 was greater than the amount forecast in 1926.

At station 56, in Newport, at the junction of the State-aid road and route 11, the routing was changed so that traffic which formerly used route 11 west now goes down the State-aid road approximately one-fourth of a mile, and then turns west. This change accounts for the increase in traffic on the State-aid road at this point. At station 11, the decrease in traffic is due to the fact that through traffic for Laconia and points north begins to divert here over South Pembroke road past Concord Airport as a short cut to route 106, avoiding the main street of Concord.

It is stated in the earlier report[3] that "It is not expected that the estimates of traffic in 1931 and in 1936 will in all cases reflect the actual traffic on each section of highway in these years, but it is believed that the estimates will reflect with reasonable accuracy ighw traffic on the trunk-line system."

Although the 1931 results have come well within the statement of expected results as above quoted, it is probable that closer estimates might have been made had gasoline consumption data for New Hampshire been available in 1926. Study of this data indicates a much closer relationship between gasoline consumption and traffic volume than between registration and traffic volume[4] particularly for the years subsequent to 1924 or 1925. The year 1924 is the first for which statistics of gasoline consumption were available for New Hampshire. When the 1926 traffic forecast was made, the gasoline consumption series for New Hampshire, as in most States, was but two years in length, too short a period to use it for forecasting purposes. The increase of 1931 traffic over 1926 traffic is 75 per cent. Gasoline consumption figures for 1931 indicate an increase over 1926 consumption in New Hampshire amounting to 73 per cent.

Special attention was paid to the occurrence of solid-tired trucks in the 1931 survey. It was found that such trucks amounted to six-tenths of 1 per cent of all trucks observed upon the highway.

A tabulation of traffic by types of vehicle for each station is given in Table 3.

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[3] Report of a Survey of Transportation on the State Highways of New Hampshire, 1927, p. 57.

[4] Report of a Survey of Traffic on the Federal-aid Highway Systems of 11 Western States, 1930.

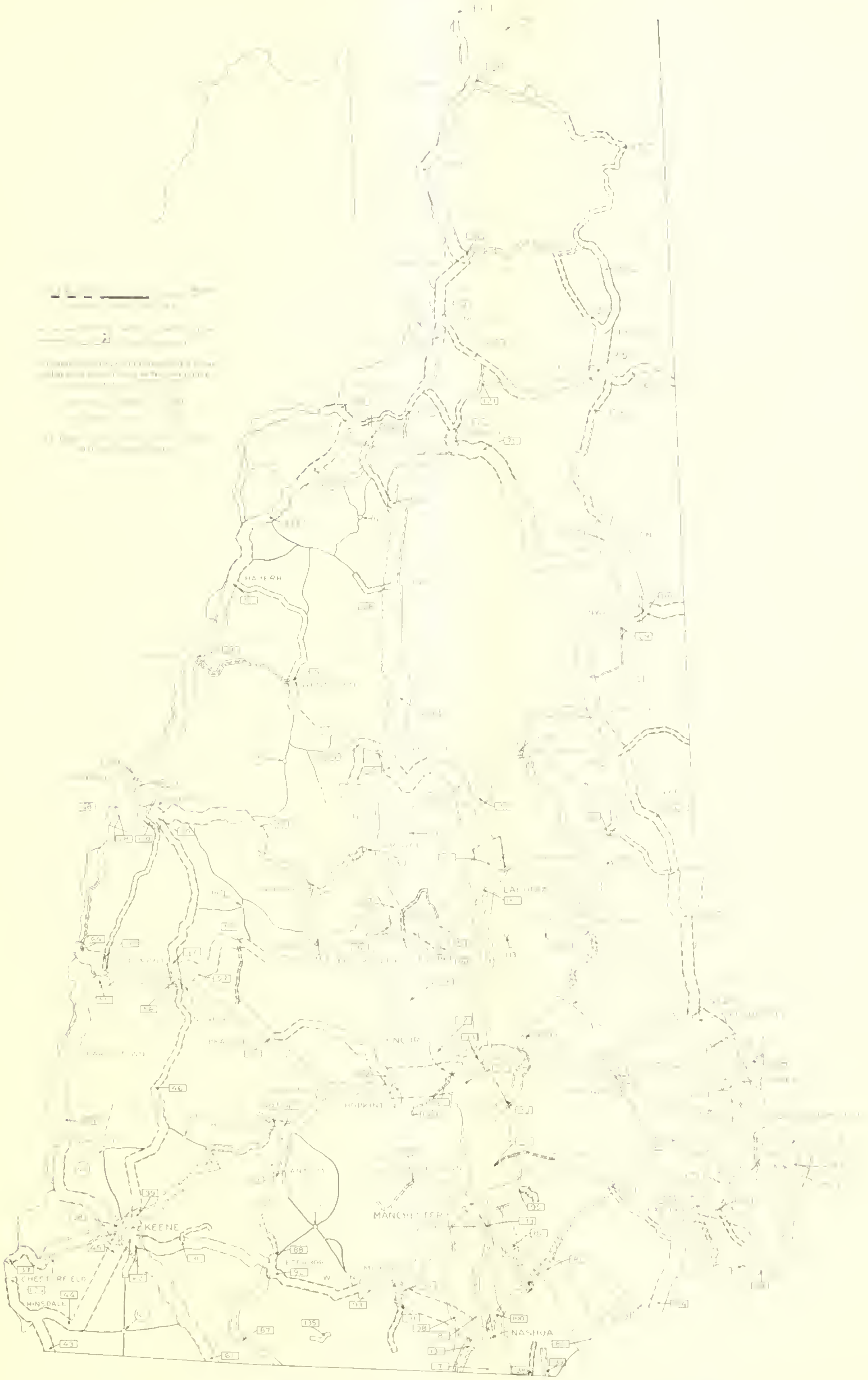

Figure 1.—Average Daily Motor Vehicle Traffic on New Hampshire Roads and Comparison with 1926 Forecast. Locations of Traffic Survey Stations Are Also Shown

TABLE 3.—*Comparison of motor vehicle traffic at New Hampshire traffic survey stations in 1931 with forecast for 1931 based on survey in 1926* [1]

Station No.[2]	Direction from station	Route No.[3]	Trucks, 1931	Passenger cars, 1931	Total vehicles, 1931	Total vehicles forecast, 1926
1	N	U. S. 1	678	10, 352	11, 030	9, 370
	S	U. S. 1	445	8, 272	8, 717	8, 190
	W	Mass. 110	107	1, 795	1, 902	1, 270
2	N	U. S. 1	366	5, 949	6, 315	8, 890
	S	U. S. 1	335	5, 914	6, 249	8, 880
	E	T. R	45	80	125	80
	W	T. R	34	112	146	80
3	N	1-A	81	4, 281	4, 362	4, 670
	S	T. R	30	376	406	1, 910
	E	1-A	214	3, 995	4, 109	2, 860
4	N	T. R	16	96	112	80
	E	U. S. 2	90	705	795	730
	W	U. S. 2	85	706	791	700
5	N	16	169	1, 854	2, 023	2, 240
	S	U. S. 2	207	2, 632	2, 839	2, 510
	W	U. S. 2	77	1, 218	1, 295	980
6	N	S. A	21	109	130	110
	E	U. S. 2	82	972	1, 054	760
	W	U. S. 2	65	1, 030	1, 095	830
7	N	U. S. 3	497	6, 504	7, 001	6, 540
	S	U. S. 3	499	6, 504	7, 003	6, 540
	W	T. R	(4)	3	3	----------
8	N	U. S. 3	442	5, 528	5, 970	5, 300
	S	U. S. 3	442	5, 528	5, 970	5, 300
9	N	U. S. 3	487	6, 196	6, 683	6, 000
	S	U. S. 3	479	6, 048	6, 527	5, 910
	W	T. R	36	232	268	160
10	N	U. S. 3	609	5, 495	6, 104	5, 580
	S	U. S. 3	548	5, 661	6, 209	5, 690
	E	101-B	51	240	291	110
11	N	T. R	89	995	1, 084	----------
	S	U. S. 3	422	5, 356	5, 778	6, 810
	W	U. S. 3	374	4, 722	5, 096	6, 810
12	N	U. S. 3	313	4, 442	4, 785	6, 560
	S	U. S. 3	318	3, 991	4, 339	6, 560
13	N	U. S. 3	230	1, 907	2, 137	3, 860
	S	U. S. 3	278	2, 238	2, 516	4, 180
	W	S. A	42	531	573	350
14	N	T. R	10	76	86	180
	E	U. S. 3	163	2, 001	2, 164	3, 650
	W	U. S. 3	199	2, 008	2, 207	3, 740
15	N	U. S. 3	257	2, 053	2, 310	3, 120
	S	U. S. 3	212	2, 105	2, 317	3, 270
	W	S. A	52	197	249	230
16	N	U. S. 3	260	3, 199	3, 459	3, 530
	S	U. S. 3	374	4, 265	4, 639	4, 020
	E	11	152	1, 557	1, 709	1, 100
17	N	U. S. 3	91	2, 051	2, 142	1, 560
	S	U. S. 3	86	1, 932	2, 018	1, 540
	E	S. A	29	426	455	310
18	N	U. S. 3	199	1, 747	1, 946	1, 770
	S	U. S. 3	198	1, 670	1, 868	1, 720
	W	T. R	9	45	54	70
19	N	U. S. 3	67	2, 510	2, 577	2, 140
	S	U. S. 3	110	2, 531	2, 641	2, 160
	E	S. A	78	185	263	220
20	N	105	31	740	771	700
	S	U. S. 3	46	1, 890	1, 936	1, 620
	E	U. S. 3	19	1, 314	1, 333	1, 030
21	N	115	40	628	668	490
	S	U. S. 3	76	1, 323	1, 399	1, 150
	W	U. S. 3	54	842	896	800
22	N	U. S. 3	92	1, 461	1, 553	1, 230
	S	U. S. 3	78	1, 375	1, 453	1, 220
	W	T R	18	154	172	20
23	N	U. S. 3	116	1, 118	1, 234	1, 300
	S	U. S. 3	119	1, 147	1, 266	1, 300
	E	T. R	8	60	68	20
24	N	U. S. 3	166	1, 465	1, 631	1, 350
	S	S. A	25	177	202	80
	E	S. A	86	563	659	230
	W	U. S. 3	92	1, 090	1, 182	1, 190
25	N	U. S. 3	96	999	1, 095	660
	S	U. S. 3	94	1, 152	1, 246	730
	W	T. R	95	714	809	310
26	N	3-A	305	2, 791	3, 096	2, 220
	S	3-A	334	3, 053	3, 387	2, 389
	W	4-B	81	696	777	480
27	S	3-A	65	1, 077	1, 142	1, 030
	E	25	90	1, 277	1, 367	1, 370
	W	25	98	1, 134	1, 232	790
28	S	T. R	145	570	715	340
	E	4	374	2, 769	3, 143	2, 280
	W	4	409	3, 199	3, 608	2, 510
28	N	"A" D. C.[5]	248	2, 269	2, 517	----------
	S	S. A	145	570	715	340
	E	D. C.[5]	374	2, 769	3, 143	2, 280
	W	"B" D. C.[5]	409	3, 199	3, 608	2, 510
29	S	10	57	856	913	810
	E	U. S. 4	154	1, 465	1, 619	1, 410
	W	U. S. 4	202	2, 149	2, 351	2, 110
30	N	U. S. 4	52	793	845	530
	S	U. S. 4	52	793	845	530
31	N	U. S. 4	100	991	1, 091	390
	E	U. S. 4 and 11	148	1, 559	1, 707	1, 080
	W	11	104	1, 006	1, 110	830
32	E	11	113	1, 221	1, 334	1, 970
	W	11	114	1, 317	1, 441	1, 970
	S	Old 11	24	154	178	----------
33	N	106	99	1, 454	1, 553	480
	E	9	112	1, 732	1, 844	1, 570
	W	9	200	3, 054	3, 254	2, 100
34	S	4-A	75	705	780	300
	E	9	83	881	964	700
	W	9	103	1, 254	1, 357	920
35	S	T. R	97	445	552	370
	E	9	258	1, 661	1, 919	1, 620
	W	9	133	1, 221	1, 354	1, 360
36	N	U. S. 4	207	2, 830	3, 037	2, 690
	S	U. S. 4	198	2, 626	2, 824	2, 450
	W	S. A	23	360	383	280
37	N	9	49	966	1, 015	850
	E	T. R	6	37	43	30
	W	9	51	997	1, 048	960
38	N	12	180	1, 949	2, 129	1, 380
	E	9 and 12	268	3, 277	3, 545	2, 370
	W	9	101	1, 489	1, 590	960
39	N	10	88	878	966	1, 230
	S	9 and 10	162	1, 910	2, 072	1, 880
	E	9	83	1, 089	1, 172	690
40	N	S. A	50	524	574	570
	E	9	76	1, 208	1, 284	1, 060
	W	9	35	764	799	490
41	N	9 and 32	95	1, 211	1, 306	850
	S	9 and 32	109	1, 268	1, 377	920
	W	T. R	16	65	81	100
42	S	S. A	104	792	896	----------
	E	9 and 103	252	4, 091	4, 343	3, 790
	W	9 and 103	229	3, 451	3, 680	3, 790
43	N. and S.	10	50	1, 173	1, 223	1, 420
44	E. and W.	10	96	1, 084	1, 180	1, 230
45	N	10 and D. C.[5]	154	1, 733	1, 887	2, 120
	S	10 and D. C.[5]	154	1, 736	1, 890	2, 120
	W	T. R	8	52	60	50
46	N	10	34	535	569	830
	S	10 and 101	39	562	601	920
	W	101	15	103	118	190
47	N	10	88	883	971	1, 010
	S	10	105	1, 053	1, 158	1, 200
	W	T. R	34	207	241	210
48	N	10	226	2, 324	2, 550	1, 650
	S	10	570	3, 087	3, 657	1, 650
49	N	10	149	1, 614	1, 763	1, 020
	S	10	192	1, 655	1, 847	1, 060
	E	T. R	33	82	115	60
50	N	10 and D. C.[5]	133	1, 583	1, 716	970
	S	10 and D. C.[5]	108	1, 428	1, 536	870
	E	111	74	476	550	190
51	N	10 and 25	129	1, 333	1, 462	1, 290
	S	10	105	1, 081	1, 186	880
	E	25	53	609	662	580
52	N	10	94	996	1, 090	1, 090
	S	S. A	41	192	233	180
	W	10	93	1, 068	1, 161	1, 110
53	N	10	134	1, 086	1, 220	970
	S	10	141	1, 403	1, 544	1, 380
	E	117	46	466	512	510
54	S	105	71	1, 015	1, 086	710
	E	10 and 18	113	1, 943	2, 056	1, 500
	W	10 and 18	171	2, 478	2, 649	2, 020
55	N	11 and 12	236	2, 549	2, 785	2, 490
	S	11 and 12	154	1, 436	1, 590	1, 280
	W	T. R	91	1, 158	1, 249	1, 310
56	S	S. A	130	1, 427	1, 557	320
	E	11	198	2, 259	2, 457	2, 070
	W	11	70	841	911	1, 770
57	N	11	118	1, 537	1, 655	1, 930
	S	32 and 103	131	1, 228	1, 359	990
	W	11 and 103	159	2, 135	2, 294	2, 040
58	S	103-A	26	308	334	180
	E	11	58	1, 005	1, 063	910
	W	11	74	1, 074	1, 148	1, 090
59	N	11 and 28	142	2, 327	2, 469	1, 160
	S	28	57	610	667	330
	E	11	108	1, 707	1, 815	1, 030
60	N	11	322	2, 671	2, 993	1, 900
	S	T. R	188	695	883	460
	E	104 and 11	489	4, 116	4, 605	2, 830
	W	104	191	1, 052	1, 243	530
61	N	12	89	1, 422	1, 511	1, 190
	S	12	90	1, 450	1, 540	1, 200
	E	T. R	4	46	50	10
62	N	12	358	2, 466	2, 824	2, 110
	S	12	210	1, 684	1, 894	1, 470
	W	T. R	157	733	890	810
63	N	12 and 101	146	1, 762	1, 908	2, 600
	S	12	78	1, 140	1, 218	2, 110
	E	101	71	648	719	570
64	N	T. R	33	145	178	470
	S	12	198	1, 544	1, 742	1, 730
	W	12	171	1, 394	1, 565	1, 180
65	N	16	363	2, 937	3, 300	3, 230
	S	16-A	266	2, 413	2, 679	190
	E	16	136	668	804	3, 060
	W	T. R	8	18	26	20
66	N	16	102	1, 271	1, 373	1, 370
	S	16	94	1, 380	1, 474	1, 350
	E	S. A	40	259	299	360

[1] Average daily traffic for the period from July 17, to Oct. 16.

[2] For locations of stations see fig. 1.

[3] The United States routes are designated by the initials U. S., State routes by number only, State-aid roads by S. A., and township roads by T. R.

[4] Less than 1 per day.

[5] Local designation.

Table 3.—*Comparison of motor vehicle traffic at New Hampshire traffic survey stations in 1931 with forecast for 1931 based on survey in 1926*—Continued

Station No.	Direction from station	Route No.	Trucks, 1931	Passenger cars, 1931	Total vehicles, 1931	Total vehicles forecast, 1926
67	N	16	44	998	1,042	930
	S	16	46	841	887	920
	E	T. R	8	60	68	30
	W	T. R	1	8	9	----------
68	N	16 and 25	98	1,297	1,395	1,160
	S	16 and 28	100	1,345	1,445	1,220
	E	25	78	565	643	520
69	N	16 and 28	70	1,901	1,971	1,340
	S	16 and 28	69	1,764	1,833	1,230
	E	T. R	14	207	221	150
70	N	16	59	1,744	1,803	1,330
	E	16	80	2,562	2,642	2,060
	W	18	82	1,900	1,982	1,440
71	N	16	33	878	911	730
	S	16	37	910	947	700
	W	T. R	13	150	163	90
72	N	16	107	840	947	910
	S	16	106	837	943	1,030
	E	T. R	30	67	97	150
	W	T. R	29	96	125	150
73	S	16	39	555	594	530
	E	26	44	608	652	410
	W	26	34	701	735	620
74	N	T. R	6	49	55	10
	E	18	101	881	982	1,010
	W	18	88	980	1,068	1,020
75	E. and W.	18	52	1,604	1,656	1,820
76	N	25	68	675	743	720
	S	25	96	864	960	800
	W	111	50	365	415	130
77	N	T. R	2	32	34	40
	S	25	170	1,667	1,837	1,620
	E	25	168	1,647	1,815	1,590
78	N	25	35	960	995	710
	S	25	83	1,460	1,543	760
	E	107	52	722	774	340
79	E. and W.	26	118	818	936	950
80	N	28	355	4,030	4,385	3,590
	S	28	275	4,028	4,303	3,600
	E	T. R	22	82	104	90
	W	T. R	80	178	258	100
81	N	28	258	2,484	2,742	3,070
	S	T. R	63	325	388	100
	E	28	262	2,201	2,463	3,020
	W	T. R	18	59	77	80
82	N	28	243	1,669	1,912	2,950
	S [6]	S. A	174	1,096	1,270	300
	E [6]	28	107	409	516	2,680
83	N	28	105	1,151	1,256	840
	S	28	102	1,138	1,240	830
	W	T. R	9	63	72	40
85	N	28	74	1,275	1,349	480
	S	T. R	22	151	173	110
	W	28	70	1,171	1,241	390
86	N	28	61	748	809	680
	S	28	90	1,187	1,277	950
	E	S. A	62	582	644	560
	W	T. R	10	44	54	20
87	N	32	77	1,107	1,184	620
	S	32	47	653	700	610
	E	32 (old)	40	625	665	10
88	N	32	85	620	705	390
	S	32	100	887	987	710
	E	T. R	43	359	402	390
89	N	32	70	673	743	400
	S	32	156	1,338	1,494	750
	W	3-A	92	728	820	390
90	N	S. A	25	330	355	----------
	S	32	64	1,102	1,166	500
	E	103	63	730	793	890
	W	32 and 103	103	1,684	1,787	1,390
91	N	S. A	33	358	391	440
	E	101	70	1,156	1,226	1,220
	W	101	92	1,419	1,511	1,590
92	N	T. R	7	48	55	80
	S	T. R	29	144	173	170
	E	101	105	966	1,071	1,010
	W	101	123	1,076	1,199	1,130
93	E. and W.	101	271	2,446	2,717	1,990
94	N	101	174	1,435	1,609	1,190
	S	101	203	1,533	1,736	1,250
	E	T. R	28	98	126	120
95	N	T. R	6	18	24	30
	S	S. A	91	636	727	640
	E	101	167	1,514	1,681	2,160
	W	101	211	2,071	2,312	2,720
96	N	T. R	2	16	18	10
	E	101	87	2,036	2,113	1,630
	W	101	85	2,016	2,101	1,630
97	N	108	157	1,391	1,548	920
	S	101 and 108	207	2,267	2,474	1,590
	E	101	142	1,366	1,508	1,110
98	N	T. R	14	80	94	70
	S	T. R	3	23	26	10
	E	101-A	267	1,917	2,184	2,470
	W	101-A	263	1,935	2,198	2,430
99	S	S. A	46	565	611	460
	E	101-C	94	1,505	1,599	1,840
	W	101-C	136	2,043	2,179	2,270
100	S	T. R	5	23	28	10
	E	102	124	804	928	470
	W	102	121	804	928	470
101	N	109	86	945	1,031	590
	E	109	64	551	615	490
	W	102	27	441	468	120
102	N. and S.	103	86	948	1,034	1,310
103	N	106	205	2,365	2,570	720
	S	106	132	1,949	2,081	450
	E	T. R	91	515	606	370
104	N. and S.	108	113	1,769	1,882	1,850
105	N	108	122	1,746	1,868	1,010
	S	108	77	1,205	1,282	740
	E	S. A	47	584	631	280
106	N	108	182	3,051	3,233	2,760
	S	108	178	2,904	3,082	2,680
	E	T. R	12	115	127	70
	W	T. R	4	47	51	50
107	E	110	132	561	693	600
	W	110	133	591	724	600
108	N	1-A	116	2,288	2,404	3,630
	S	1-A	116	2,290	2,406	3,550
	W	T. R	57	737	794	480
109	N. and S.	3-A	134	1,560	1,694	1,650
110	N	18	51	905	956	1,260
	E	18 and 28	81	1,316	1,397	1,510
	W	T. R	43	630	673	600
111	N	S. A	51	416	467	240
	S	S. A	57	478	535	290
	W	T. R	23	153	176	60
112	N. and S.	S. A	41	328	369	240
113	N	T. R	8	93	101	140
	S	T. R	17	170	187	110
	E	T. R	19	124	143	60
	W	T. R	16	160	176	100
	NW	T. R	22	176	198	70
115	N	T. R	35	160	195	80
	E	S. A	40	245	285	190
	W	T. R	15	113	128	110
116	N	T. R	5	46	51	20
	S	T. R	3	31	34	30
	E	T. R	7	60	67	60
	W	S. A	6	56	62	60
117	N	S. A	50	601	651	180
	S	S. A	50	561	611	----------
	E	S. A	33	159	192	130
	W	S. A	17	65	82	80
118	N	S. A	17	46	63	50
	S	S. A	15	51	66	80
	W	S. A	7	8	15	10
119	N	S. A	47	117	164	140
	S	S. A	47	120	167	140
	E	T. R	2	14	16	10
120	N	116	60	725	785	580
	S	T. R	24	285	309	250
	W	116	45	484	529	380
121	N	T. R	27	241	268	210
	S	115	36	450	486	410
	E	115	32	339	371	250
122	N	T. R	8	42	50	80
	S	S. A	9	57	66	120
	E	T. R	5	36	41	60
123	N	T. R	(4)	4	4	10
	E	T. R	9	71	80	100
	W	T. R	9	69	78	90
125	N	T. R	7	111	118	160
	S	S. A	7	79	86	90
	E	S. A	10	150	160	170
	W	S. A	11	135	146	120
126	N	T. R	7	34	41	40
	S	T. R	5	34	39	40
	E	T. R	2	5	7	----------
127	N	S. A	5	40	45	30
	S	112	15	131	146	120
	E	112	17	130	147	110
128	E. and W.	112	34	540	574	610
129	S	T. R	57	201	258	280
	E	S. A	150	1,410	1,560	1,120
	W	S. A	193	1,583	1,776	1,280
130	N	10-A	71	531	602	570
	S	10-A	67	505	572	540
	W	T. R	12	61	73	60
131	N	S. A	14	49	63	50
	S	S. A	12	50	62	60
	E	T. R	8	14	22	----------
132	N	T. R	32	132	164	110
	S	T. R	48	263	311	220
	E	114	166	1,602	1,768	1,190
	W	114	141	1,358	1,499	1,020
133	N. and S.	S. A	81	930	1,011	200
134	N	T. R	29	451	480	250
	S	S. A	70	689	759	370
	E	T. R	25	238	263	140
	W	S. A	38	310	318	270
	SW	T. R	38	383	421	30
135	E. and W.	S. A	77	417	494	480
136	N	S. A	107	719	826	590
	S	S. A	77	622	699	490
	E	T. R	40	139	179	140

[4] Less than 1 per day.

[6] Route 28 under construction for a distance of 3.7 miles during 1931 survey. Traffic was detoured south over State-aid route to route 102 during this time.

TABLE 3.—*Comparison of motor vehicle traffic at New Hampshire traffic survey stations in 1931 with forecast for 1931 based on survey in 1926*—Continued

Station No.	Direction from station	Route No.	Trucks, 1931	Passenger cars, 1931	Total vehicles, 1931	Total vehicles forecast, 1926
137	S.	S. A.	103	700	803	950
	E.	S. A.	262	1,647	1,909	1,210
	W.	T. R.	161	1,016	1,177	630
138	N.	S. A.	59	346	405	490
	S.	S. A.	60	350	410	490
	W.	T. R.	7	41	48	40
139	N. and S.	S. A.	64	650	714	370
140	N.	S. A.	55	462	517	430
	S.	S. A.	47	425	472	350
	E.	T. R.	11	58	69	80
	W.	T. R.	2	14	16	20
141	N. and S.	S. A.	20	118	138	210
142	N.	S. A.	33	200	233	210
	S.	S. A.	29	159	188	150
	W.	T. R.	8	51	59	60
143	N.	S. A.	167	1,717	1,914	1,500
	E.	S. A.	84	1,064	1,148	2,670
	W.	S. A.	98	765	863	1,230
144	N.	T. R.	147	1,239	1,386	1,230
	S.	T. R.	23	191	214	70
	E.	T. R.	33	271	304	230
	W.	S. A.	78	722	800	510
	SW.	T. R.	87	873	960	970
145	N.	T. R.	16	158	174	200
	S.	T. R.	20	169	189	200
	E.	T. R.	21	180	201	250
	W.	T. R.	22	165	187	220
146	N.	10-A	63	343	406	530
	S.	10-A	64	340	404	540
	E.	T. R.	4	11	15	20
	W.	T. R.	7	16	23	------
147	N.	T. R.	13	64	77	80
	S.	T. R.	1	12	13	20
	E.	T. R.	12	72	84	110
	W.	T. R.	23	128	15	190

(Continued from p. 130)

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CEMENT CONCRETE GIVES GOOD SERVICE ON CONNECTICUT AVENUE EXPERIMENTAL ROAD

Three sections of the Connecticut Avenue experimental road (experiments 3, 4, and 5, north of Bradley Lane) were built of Portland cement concrete. A portion of the work was done in the fall of 1912, and the sections were completed in May, 1913. The photographs shown on the covers of this issue were taken on experiment 5. The back cover shows the method used in construction and the appearance of the pavement about a year after completion. The front cover shows the appearance of a typical section in August, 1932.

The concrete on experiment 5 was composed of a 1:1¾:3 mix. Gravel, limestone, and trap rock were used as coarse aggregate in different sections of the experiment. The thickness of the pavement is 6 inches throughout. The cost of the surface was 142.29 cents per square yard. In 1922 this experimental section was surface-treated with a hot application of 0.26 gallon of refined coal tar and pea gravel. This mat surface has now entirely disappeared.

The subgrade on which the Connecticut Avenue experimental road was constructed is a silty loam with varying percentages of mica. Excessive mica causes detrimental elasticity in soils, and this fact may account in part for the large amount of cracking which has occurred in the concrete sections.

It is also believed that defective drainage has had a great deal to do with the deterioration of the concrete. In February, 1929 (see PUBLIC ROADS, vol. 11, No. 4, June, 1930), immediately after a heavy rain, it was noted that a considerable volume of water was flowing to the surface of the concrete sections through the cracks. This condition was found throughout the concrete portion of the experiment, although it was most severe on experiment 3. It was also observed that the sewers were not receiving any flow from either side drain or through the cross drains. Upon digging down into the French drains it was found that wherever one of the utility companies had made a cut through the drain line, it had failed to replace either tile or stone and had backfilled the trench with earth, thereby effectively destroying the drainage system. It was learned from the records of the sanitary commission that most of the service connections were made in 1922. It is also a fact that the cost of maintaining the concrete sections has materially increased since that year. The total cost of maintenance to date, including the surface treatment in 1922, amounts to 37.71 cents per square yard.

The concrete portion of the Connecticut Avenue experimental road can not be regarded as a particularly successful example of concrete road construction even for the early day in which it was built; and great advances have been made, in both knowledge and technique, since that time. However, in spite of the excessive cracking in some areas, the pavement is still in service under an average traffic of about 3,000 vehicles per day.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

SEPTEMBER 30, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
					Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$					$	$				$	
Alabama	2.418.7	1.257.811.92	607.009.92	99	49.3		49.3	2.170.626.64	1.085.313.26	63.4	33.0	96.4	4,385.699.70	Alabama
Arizona	1.254.6	2.700.914.60	1.631.424.04	64	65.8	97.4	163.2	583.308.09	123.802.52	30.1	30.6	60.7	231,407.42	Arizona
Arkansas	1.967.2	2,262,708.94	1.033.985.95	85	64.5	38.4	102.9	362,032.53	181,016.26	17.3		17.3	2,183.603.25	Arkansas
California	2.410.4	7.547.903.09	3.594.088.12	67	176.6	39.3	215.9	985.578.29	60,131.16	12.7	7.6	20.3	459.301.63	California
Colorado	1.615.4	3.839,201.88	2.031.886.03	89	148.8	68.6	217.4						1.788.004.65	Colorado
Connecticut	287.0	4.071.872.74	1.479,495.06	77	24.9		24.9	938,125.66	441,086.28	15.1		15.1	225.940.18	Connecticut
Delaware	367.9	797.087.35	358.463.00	90	22.1	2.0	24.1	13.166.40	2,500.00	.2	.1	.3	139.440.82	Delaware
Florida	660.2	3.668.655.29	1.696,567.65	72	105.0		105.0	88.745.83	44.372.91	4.7		4.7	2.505,435.86	Florida
Georgia	3.159.4	5.742.147.73	2.593.600.14	73	123.1	147.5	270.6	566.378.33	241.004.44	25.2	3.9	29.1	781.344.34	Georgia
Idaho	1.260.2	3.200.934.38	1.879.704.51	84	134.4	315.7	450.1	665.214.83	170.816.10	39.6	48.4	88.0	304.388.69	Idaho
Illinois	2.777.7	24.390.834.30	11.220.790.34	86	738.0	37.5	775.5	4.110.799.43	953.338.61	115.2	5.2	120.4	715.060.85	Illinois
Indiana	1.883.2	10.060.334.77	4.997,511.19	83	376.2	15.9	392.1	316.400.18	158.198.94	16.0		16.0	339,387.72	Indiana
Iowa	3.297.5	7.946.729.37	2.667.207.51	76	384.6	89.9	474.5	1.220.311.26	22.923.25	50.5	1.2	51.7	9.944.78	Iowa
Kansas	3.549.4	5.041.360.98	2.458.180.14	69	240.2	237.3	477.5	2,321.436.18	664.103.29	158.3	53.6	211.9	304,715.61	Kansas
Kentucky	1.820.5	3.596.756.19	1,418,380.52	61	161.9	108.6	270.5	1.896.177.98	637.787.66	76.7	60.2	136.9	468.144.52	Kentucky
Louisiana	1.569.2	7.109.560.16	3.296,068.98	52	86.1	9.9	96.0	1.083.662.52	278.102.31	27.4	4.5	31.9	513.742.69	Louisiana
Maine	748.7	2.629.773.58	1.136.825.80	84	89.2		89.2	495.904.06	111.099.78	23.1		23.1	104.340.95	Maine
Maryland	794.5	1.729.540.73	754.093.93	57	66.3	4.6	70.9	145.106.45	51,453.72	5.4	1.5	6.9	27,400,49	Maryland
Massachusetts	821.6	6.819.080.31	2.588.009.96	85	72.3	6.6	78.9	1.076.511.01	264.318.57	13.4	2.1	15.5	130,743.13	Massachusetts
Michigan	2.156.2	8.285.935.23	3.618.443.29	73	311.5	75.8	387.3	1.501.894.90	521.455.00	61.3	15.2	76.5	1.031,277.36	Michigan
Minnesota	3.549.9	14,620.610.63	3.960,301.38	93	267.9	403.0	670.9	1.521.934.97	10.600.00	69.0	32.4	101.4	8.400.00	Minnesota
Mississippi	1.818.2	3.952.195.21	1.936.470.53	83	201.8	73.1	274.9	135.336.30	67.668.14	4.4	.2	4.6	4.924,347.04	Mississippi
Missouri	3.033.6	5.008.894.87	2.116.809.03	76	124.8	61.2	186.0	1.160.882.81	243.619.71	68.0	5.0	73.0	459,932.21	Missouri
Montana	2.727.3	4.508.846.58	2.541.932.75	75	399.9	105.4	505.3	1.763.307.02	939.824.58	97.6	177.2	274.8	1,758,180.06	Montana
Nebraska	4.220.8	3,446.743.26	1.676.726.38	58	121.7	116.3	238.0	1.188.761.70	561.820.82	31.4	38.8	70.2	1,508.591.11	Nebraska
Nevada	1,328.6	1,963.828.00	1.647.258.61	78	38.4	125.2	163.6	175.911.80	70.528.72		35.2	35.2	71,286.33	Nevada
New Hampshire	425.4	695.726.79	318,698.36	43	23.3	2.8	26.1	29,252.65	11.701.06	1.5		1.5	277,434.90	New Hampshire
New Jersey	623.6	6,285.477.62	2.329.616.22	52	38.9	.5	39.4	1.212.488.64	303.122.19	23.0		23.0	117,276.97	New Jersey
New Mexico	2.222.1	2,004.098.43	1.354.606.04	80	84.9	45.9	130.8	1.033.921.62	426,786.90	50.3	49.7	100.0	727.640.42	New Mexico
New York	3,280.8	19.581.650.00	8.133.273.00	60	539.1	17.7	556.8	6.145,800.00	1.030,125.00	104.0	11.4	115.4	1,222,104.60	New York
North Carolina	2,222.4	1.916,337.19	955.341.85	58	151.9	9.0	160.9	1.428.465.12	719.502.07	191.9	17.2	209.1	2,771.461.76	North Carolina
North Dakota	4.951.7	4.540.483.91	2,056,995.48	66	510.8	487.3	998.1	1.766,172.72	814,176.63	74.3	301.7	376.0	208.600.79	North Dakota
Ohio	2.857.3	11.546.103.40	3.822.167.43	64	274.7	91.2	365.9	1.202.537.00	265,168.52	24.9	10.9	35.8	809.740.59	Ohio
Oklahoma	2,333.5	4,037.930.62	1.983,608.83	75	212.1	43.5	255.6	1.044,113.36	563.526.09	47.5	10.8	58.3	874,482.27	Oklahoma
Oregon	1.515.4	3.527,380.58	1.885.614.10	77	130.9	96.5	227.4	607,931.83	198.214.33	20.4	44.9	65.3	730.581.32	Oregon
Pennsylvania	3.052.4	10.113.662.29	4.555.059.33	60	331.0		331.0	2,379.893.71	703.488.85	71.3		71.3	965.356.01	Pennsylvania
Rhode Island	258.2	1.170.799.67	357.950.47	70	20.7	4.3	25.0						260.037.51	Rhode Island
South Carolina	2.043.9	2.656.702.92	1.162.269.32	91	60.6	46.8	107.4	1,374.216.08	465,354.00	124.3	54.0	178.3	184.493.25	South Carolina
South Dakota	3.923.1	5.366.921.59	2,588,066.63	60	424.3	311.2	735.5	116.382.69	51.770.14	21.4	9.9	31.3	152,258.70	South Dakota
Tennessee	1.693.1	2.016,384.60	1.004.738.14	46	68.5	21.8	90.3	2,042.411.29	1.021,205.58	84.9	7.5	92.4	1.814.549.10	Tennessee
Texas	7.809.5	14.568.472.31	6.519.508.53	70	727.0	292.0	1.019.0	4.769.118.56	1.782.682.61	186.6	223.6	410.2	1.798.044.54	Texas
Utah	1.123.5	1.578.866.18	1.117.471.68	74	95.5	134.6	230.1	768,585.66	208,748.74	46.8	5.1	51.9	415.392.45	Utah
Vermont	352.5	819,001.36	386.890.83	68	32.3		32.3	80.521.43	7.246.93	7.0		7.0	46.423.32	Vermont
Virginia	1.917.9	2.534,859.55	1.230,847.77	67	143.8	23.3	167.1	1.208,197.34	533.445.09	58.2	7.9	66.1	1.204,351.54	Virginia
Washington	1.236.3	2.790.083.63	1.275.720.63	87	97.3	7.4	104.7	1,054.412.78	262.423.12	38.7	1.1	39.8	572,278.77	Washington
West Virginia	899.8	2.304.566.52	897.622.54	76	61.4	5.0	66.4	1.894,163.38	620.267.41	69.9	8.1	78.0	315.328.26	West Virginia
Wisconsin	2.559.3	9.805.172.92	3.586,122.02	87	263.8	114.8	378.6	790.708.51	88.000.00	25.2	25.6	50.8	151.382.52	Wisconsin
Wyoming	1.950.0	3.439.473.97	2.144.615.04	88	187.6	215.1	402.7	211.017.21	86.284.57	25.4	5.4	30.8	25.683.05	Wyoming
Hawaii	74.8	1.442.767.53	920.136.34	43	40.6	1.7	42.3	651,487.23	324,288.62	15.3	1.9	17.2	1.050.166.62	Hawaii
TOTALS	101.124.4	260.943.185.67	115.528.175.34	73	9.116.3	4.151.6	13.267.9	58.319.334.20	18.394.414.48	2.338.8	1.352.6	3.691.4	42.075.130.65	TOTALS
Construction completed		183.569.000.00	84.645.000.00											
Balance uncompleted		77.374.000.00	30.883.000.00											

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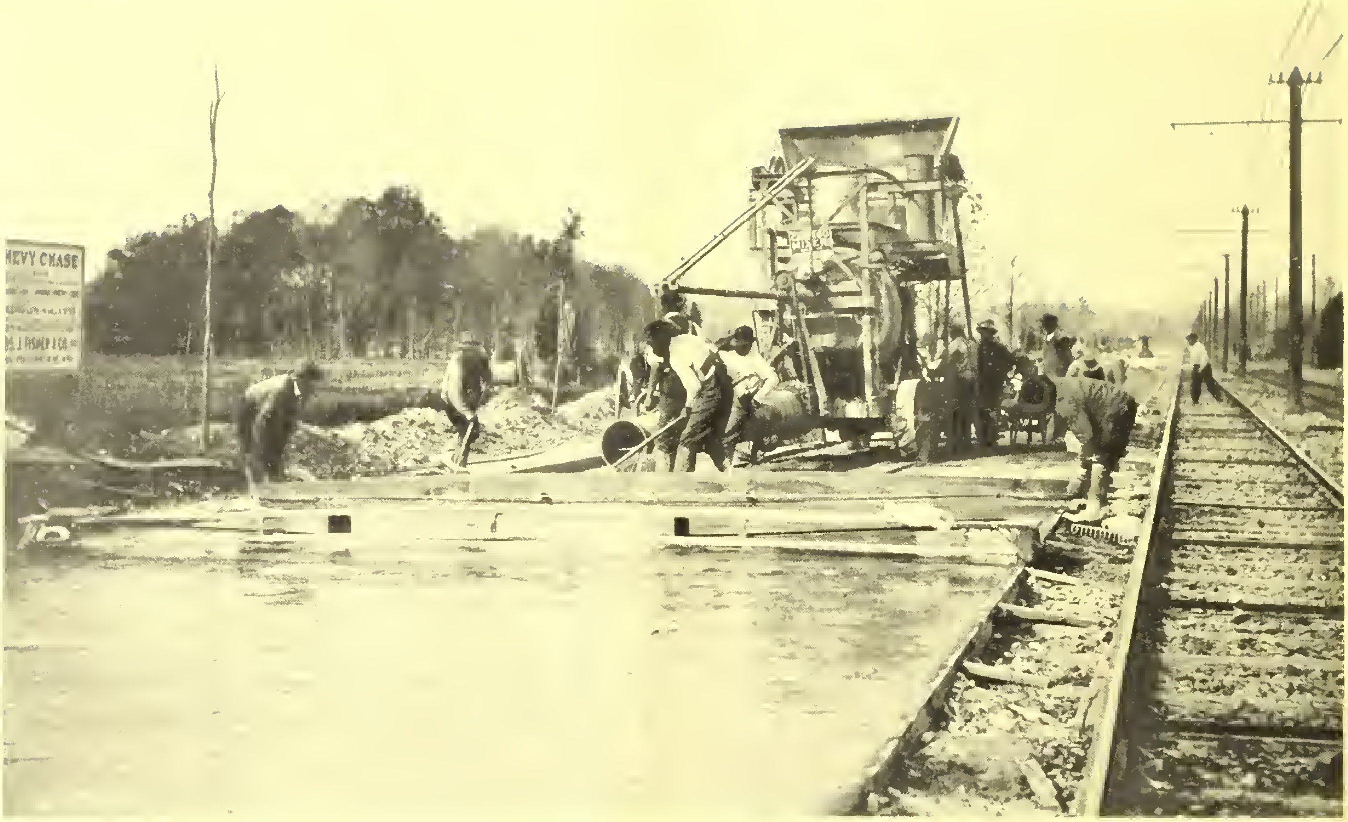
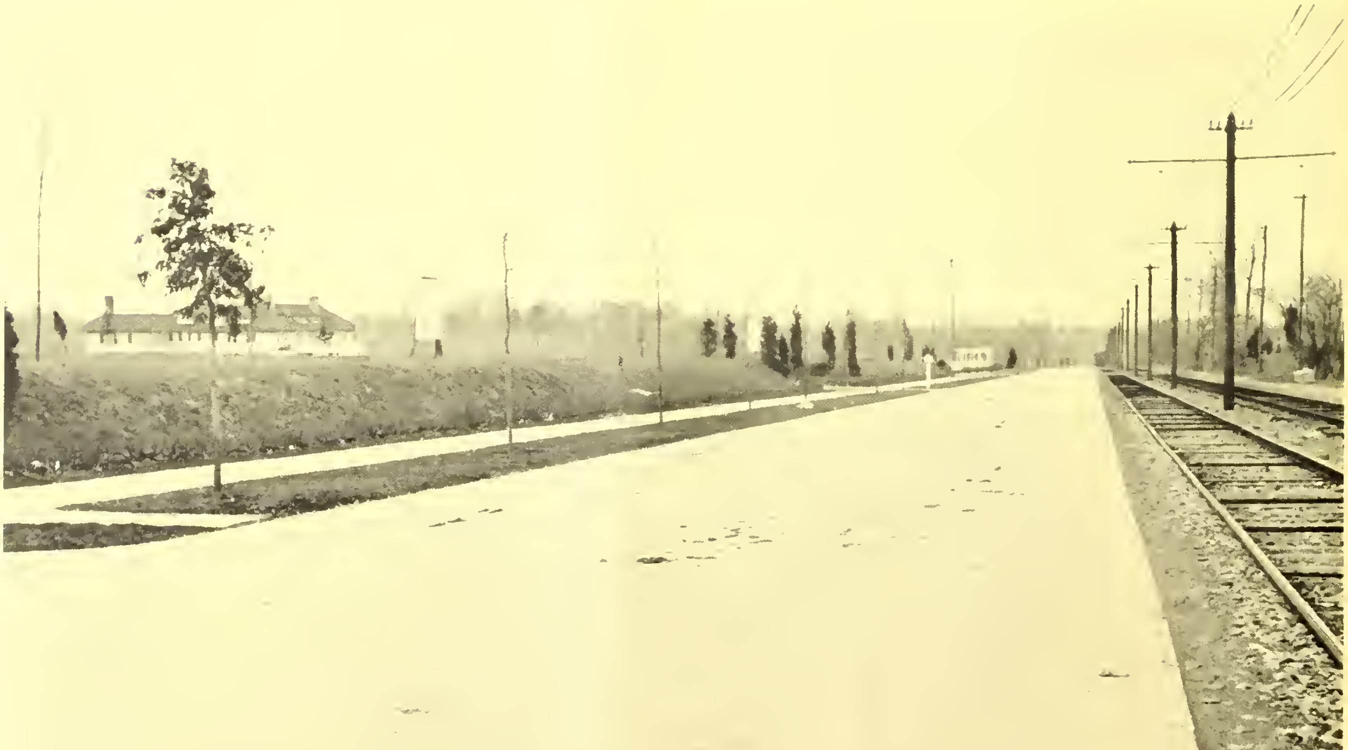
[1] The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.



CONCRETE CONSTRUCTION ON CONNECTICUT AVENUE EXPERIMENTAL ROAD, NOVEMBER, 1912



CONDITION OF CONCRETE SURFACE IN APRIL, 1914

U. S. GOVERNMENT PRINTING OFFICE 1932

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 9 ▽ NOVEMBER, 1932

A BRICK PAVEMENT 19 YEARS AFTER ITS CONSTRUCTION

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 9 November, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

ROAD IMPACT PRODUCED BY A HEAVY MOTOR BUS

Reported By **JAMES A. BUCHANAN**, Associate Engineer of Tests, United States Bureau of Public Roads

DETERMINATION of the magnitude of the impact forces exerted on the pavement by the wheels of motor trucks and busses has been the object of an extended series of investigations by the Bureau of Public Roads, in cooperation with the Rubber Manufacturers' Association and the Society of Automotive Engineers. In the course of these investigations the major factors which affect the magnitude of such impact reactions have been separately and collectively studied both in the field and in the laboratory. In order to round out this research a final series of tests has been made with a bus chassis capable of carrying heavy loads at high speeds, to anticipate, in some measure at least, the trend in operating conditions for several years to come.

CONCLUSIONS DERIVED FROM BUS IMPACT TESTS

1. With balloon tires as used in these tests, reactions of about three times the static wheel load may be encountered in isolated cases, generally where pavement failure has already occurred and repairs are necessary.
2. A few reactions amounting to twice the static wheel load may be expected for each mile of rough surface.
3. The frequent reactions for rough surfaces amount to about 1.5 times the static wheel load.
4. Even the smoothest roads produce frequent reactions of from 1.1 to 1.2 times the static wheel load.
5. Concerning the maximum reactions produced at a large number of individual roughness conditions, the general tendency is for this maximum to increase in approximate proportion to speed up to about 40 miles per hour, and at speeds greater than that comparatively slight increases in reaction are noted. However, occasional roughness conditions may occur where the maximum reaction developed at 20 or 30 miles per hour is as great as the maximum produced by the majority of the roughnesses at only the higher speeds.

PURPOSE OF THE TESTS

This series of tests was planned with three objects in mind. First, data were desired concerning the magnitudes of the impact reactions produced at high operating speeds and the relative magnitudes of the shock and drop types of reaction throughout the full range in speed represented by modern operating conditions. Second, more complete data were desired concerning the magnitude of the reactions developed by a considerable number of severe but typical roughness conditions occurring on the surfaces of highways in actual use. Third, data were desired concerning the relation between the magnitude of reaction and the frequency of occurrence for the full range of surface roughness of highways. The data obtained in these tests and the conclusions drawn therefrom are presented herewith.

HEAVY-DUTY BUS SELECTED

The vehicle selected for use in these tests was a 240-inch wheel-base, heavy-duty bus chassis. It was designed to have a gross weight of 24,000 pounds, and this was obtained in the tests by placing iron weights on a special platform body, as shown in Figure 1. The front-axle load was 8,000 pounds and was carried by 9.75–24 inch balloon tires throughout the tests. The rear-axle load of 16,000 pounds was carried on interchangeable 9.75–24 inch dual and 12.75–24 inch single balloon tire equipments. The respective tires were inflated to pressures recommended by the Tire and Rim Association for the capacity load carried.[1] The vehicle had an available speed range of 0 to 80 miles per hour. The average unsprung weight at the right rear wheel was 1,817 pounds.

STATIC TESTS

The tires were subjected to the usual tests under static load conditions. The curves of static load versus vertical deformation for the single and dual tire equipments used on the rear wheels are given in Figure 2. It has been the experience throughout all the tests that dual tires of a given type are somewhat less cushioning than single tires of the same type and of corresponding carrying capacity. The reason for this is found in the respective cross-sectional dimensions of the two types of mounting. A single tire has greater sectional depth than dual tires of the same type and carrying capacity. This greater depth permits greater vertical deformations which decrease the abruptness of velocity changes or accelerations imparted to the axle and correspondingly decrease the dynamic components of the impact reactions.

In Figure 3 data are given concerning the areas of contact between the tires and a plane reacting surface under static loads. The area of contact is taken as the entire area within the outline of the impression. The broken line in this figure represents the entire area under the influence of the dual tires. This gross area was obtained by multiplying the length of a given contact impression by the center to center spacing of the dual tire mounting and adding to this product the contact area for one of the tires in the dual mounting.

Data concerning the static tests of the tires are given in Table 1. The areas of contact are included because they have a bearing on the stresses developed within the pavement structure.

TABLE 1.—*Data from static tests of tires*

Tire size .... inches..	9. 75–24	12. 75–24
Mounting	Dual.	Single.
Plies	10	16
Rim size .... inches..	24 x 8	24 x 11
Dual spacing .... do....	12	....
Inflation pressure .... lbs. per sq. in..	63	90
Vertical deformation:		
8,000-pound wheel load .... inches.	1. 23	1. 49
16,000-pound wheel load .... do....	2. 17	2. 62
24,000-pound wheel load .... do....	3. 08	3. 72
Gross contact area [2]:		
8,000-pound wheel load .... sq. ins..	192	92
16,000-pound wheel load .... do....	282	162
24,000-pound wheel load .... do....	354	211
Average unit pressure based on gross contact area:		
8,000-pound wheel load. lbs. per sq. in..	42	87
16,000-pound wheel load .... do....	57	99
24,000-pound wheel load .... do....	68	114

INSTRUMENTATION DESCRIBED

As in former tests, the method of determining the magnitudes of the impact forces was to measure the accelerations imparted to a rear wheel by means of contact-type accelerometers rigidly mounted on the

---

[1] The inflation pressures were determined with a calibrated Bourdon-tube pressure gage. Frequent observations of inflation pressures were made throughout the tests.

[2] Gross contact area for dual tires includes area between the tires. It is obtained by multiplying the contact length by the dual spacing and adding the area of contact of a single tire.

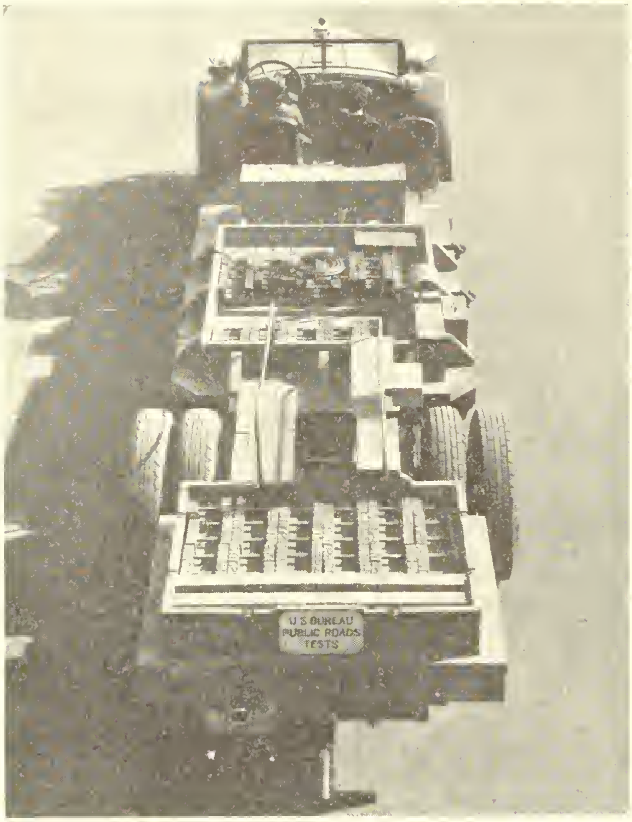

Figure 1.—The Bus Chassis as Used in the Tests, Showing the Arrangement of 100-Pound Weights

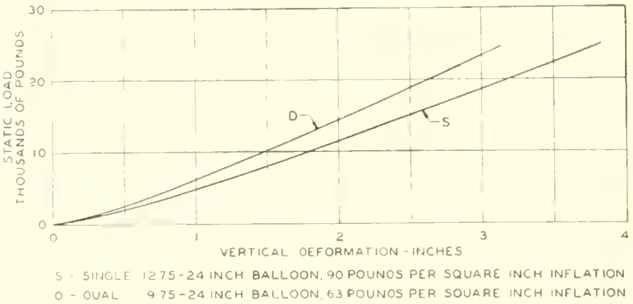

Figure 2.—Static Load-Deflection Curves of the Tires

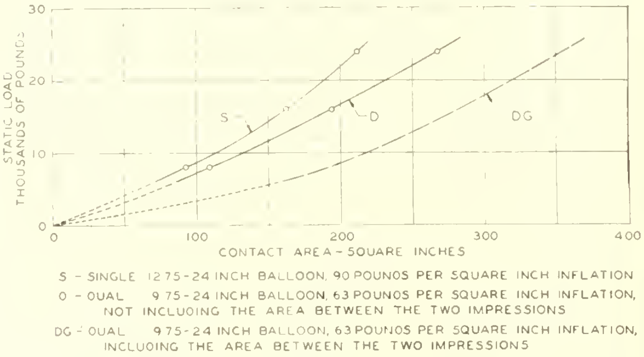

Figure 3.—Static Load-Contact Area Curves of the Tires

rear axles at the saddle which received the vehicle spring. The principle involved in the use of these contact accelerometers is briefly described as follows:

A pivoted weight or inertia element is restrained from oscillation, except through a minute arc, by two adjustable screws which also constitute electric contacts. A calibrated cantilever spring is attached to the pivoted weight. This spring is deflected known amounts by a micrometer adjustment at its outer end. The pressure imposed on the spring causes the inertia element to exert a known force against one of the contact stop screws. The accelerometer unit, being rigidly fastened to the vehicle axle, receives the accelerations imparted to the axle. When the product of the imparted acceleration and the mass of the inertia element is a force greater than the force due to the deflected cantilever spring, the inertia element will break away from one contact stop screw, travel not more than two-thousandths of an inch and make contact with the other contact stop screw. Whether or not this movement takes place, i. e., whether or not the imparted acceleration exceeds the acceleration corresponding to a given deflection at the micrometer adjustment of the cantilever spring, is determined by an autographic record or by means of a telephone circuit.

The 10-element, autographic, contact-type accelerometer was fully described in an earlier report on impact tests which appeared in the April, 1931, issue of Public Roads. This instrument, which is shown in Figure 4, was used in the tests involving artificial obstructions and natural roughness conditions. Because of the great length of records which would have been involved, the autographic instrument was not used in the tests conducted over the half-mile lengths of typical road surfaces. For these latter tests a single-element contact accelerometer recording through telephones was used.

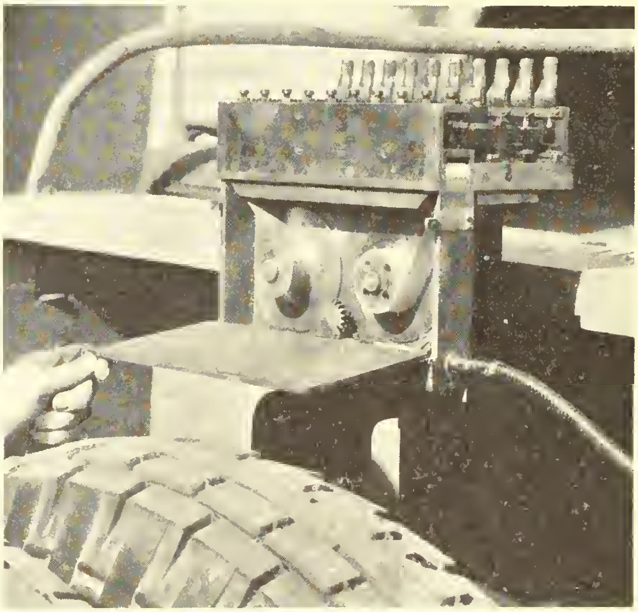

Figure 4.—The 10-Element Autographic Contact Accelerometer Mounted at the Rear Wheel. The Open Cover Shows the Method of Drawing Special Paraffin-Coated Paper Beneath the Styli Attached to the Accelerometer Elements. The Flexible Shaft at the Right Contains a Cable Driven by an Electric Motor

The single-element, telephone-recording instrument, which is shown as used in the tests in Figure 5, was described in the July, 1930, issue of Public Roads, in an article dealing with the instrumentation involved in motor vehicle impact tests. There was the following modification in the method of observing the action of the single-element accelerometer. The telephone circuit was arranged with three head sets in parallel which were used by three observers simultaneously. In this

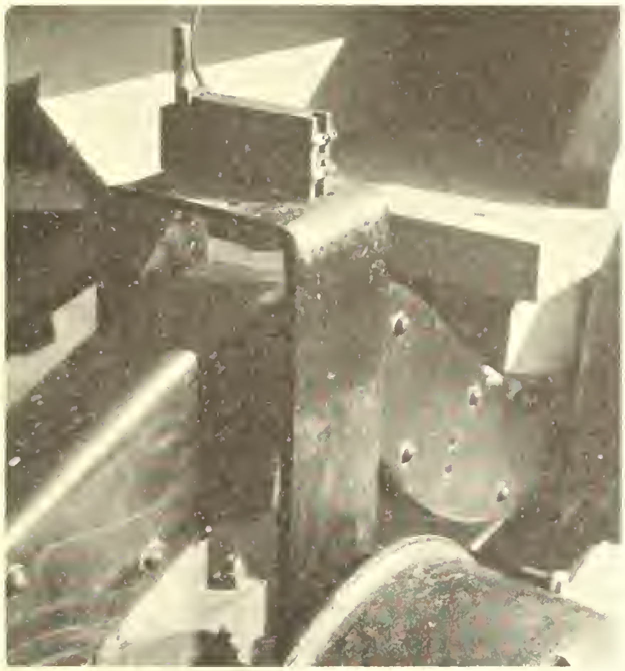

FIGURE 5.—THE SINGLE-ELEMENT CONTACT ACCELEROMETER MOUNTED ON THE YOKE WHICH IS BOLTED TO THE REAR AXLE AT THE SPRING PAD

way a check was obtained on the counts of the number of times an acceleration of a given magnitude was imparted to the axle during the course of a test run. By making a number of runs for which the acceleration setting was successively increased until no response was recorded, the desired data on acceleration versus frequency of occurrence were readily obtained for each road test condition.

The impact force was obtained in the usual manner by multiplying the mass of the unsprung weight at the rear wheel by the acceleration as determined in each test and adding to this product (which is the dynamic increment) the static weight on the wheel.[3] The flexing of the vehicle spring above or below its static load position may cause minor variations of the actual total force from the force as computed by this method. It has been pointed out in the earlier publications, however, and justified by experimental evidence, that the complication of the test procedure involved in securing data to make a correction for this variation is not warranted by the nature of the tests. The average pressure exerted by the spring is approximately that of the static load position. The contact accelerometer is not particularly adapted to the task of obtaining a simultaneous record of the flexing of the vehicle spring, and, in the interest of simplification of instrumentation and test procedure such corrections have not been made.

What the nature of the corrections would be in the case of the artifical obstructions has been clearly indicated by the truck impact tests described in PUBLIC ROADS, volume 7, No. 4, June, 1926. In these tests the sprung component of the impact force was measured. In the case of shock it appears, from both theory and observation, that the maximum impact takes place at a point when the axle is only slightly lifted above its normal rolling position, and for this reason the increase in spring pressure under shock conditions is negligible.

In the case of drop after striking an artificial obstruction it was found that in practically all cases the spring pressure was reduced below its value under static conditions. The explanation of this phenomenon appears to be about as follows. The body, as a result of the force applied by the shock, starts upward in a swing which is of relatively long duration compared to the more violent oscillations of the axle. At the instant of maximum drop impact the body has very likely not returned to its static level. At the same time the axle has been depressed below its static position by an amount equal to the increased deflection of the tire under the drop impact. The result is a reduction of the spring pressure. With very light loads it has been found that this parting of the wheel and body is at times so great as to put the spring in tension. With heavy loads, such as those used in the bus tests, this does not occur, but a considerable reduction of the spring pressure may take place.

Had the spring deflections been measured in the tests with artificial obstructions the impact forces shown in Figures 6 and 7 would have been less than those plotted. In the case of natural obstructions it is impossible to determine in what phase of their relative motion the sprung and unsprung masses may be at the instant of impact. The spring pressure under static load has therefore been taken as representing a fair average for all road conditions.

TESTS INVOLVING ARTIFICIAL OBSTRUCTIONS

The data obtained in that portion of the tests which involved the artificial obstructions are given in Figures 6 and 7. In addition to yielding information concerning the relative magnitudes of shock and drop reactions at speeds up to 75 miles per hour these tests serve as a tie-in to the earlier impact investigations where artificial obstructions were used on an otherwise smooth, level road surface.

Three steel obstructions were used. The 1½ by 30 inch inclined plane and the 1½ by 12 inch rectangular obstruction were the same as those used in previous tests, while the 1 by 12 inch rectangular obstruction was introduced in these tests. The method of securing them in place was by bolting them to nuts set in the concrete pavement, and only one obstruction was placed on the road at any time. Data for both shock and drop conditions were taken with all three obstructions.

The tests involving the use of artificial obstructions were made on a completed section of the Mount Vernon Memorial Highway before it was opened to general traffic. It was fortunate that a portion of this smooth, wide highway could be made available for this work and provisions made for controlling all other traffic while the tests were in progress. The vehicle used had a gross weight of 12 tons and, although speeds of about 80 miles per hour, or 120 feet per second, were attained, the tests were conducted without danger to life or property.

In Figure 6 the data are given for the tests with the dual 9.75–24 inch balloon tires over the three artificial obstructions and Figure 7 gives the corresponding data for the single 12.75–24 inch balloon. The inflation pressures were maintained at 63 and 90 pounds per square inch, respectively, in these and all subsequent tests.

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[3] This is expressed by the eqnation

$F=mA+L$,

in which F=total vertical reaction per rear wheel.
m=mass of unsprung weight per rear wheel.
A=acceleration imparted to nnsprung mass.
L=static load on pavement, per rear wheel.

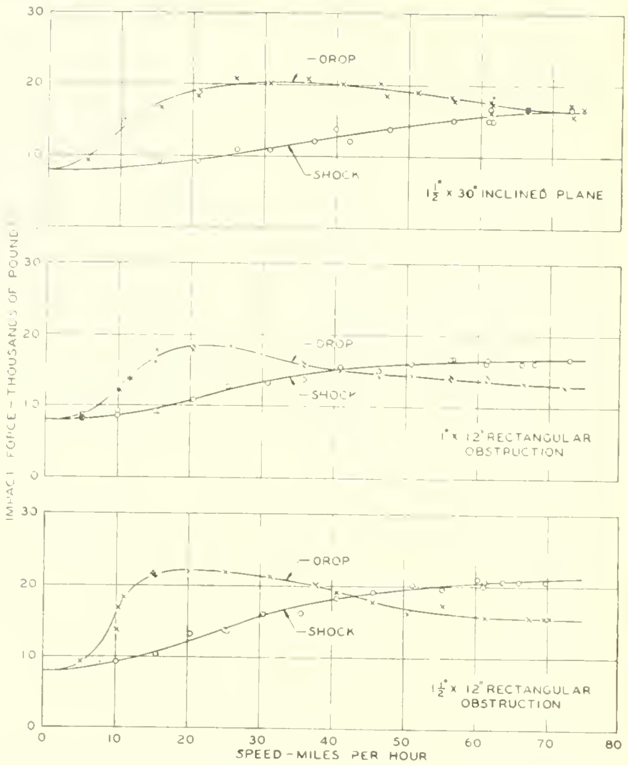


Figure 6.—Impact Tests with Artificial Obstructions and Dual 9.75-24 Inch Balloon Tires at 63 Pounds per Square Inch Inflation

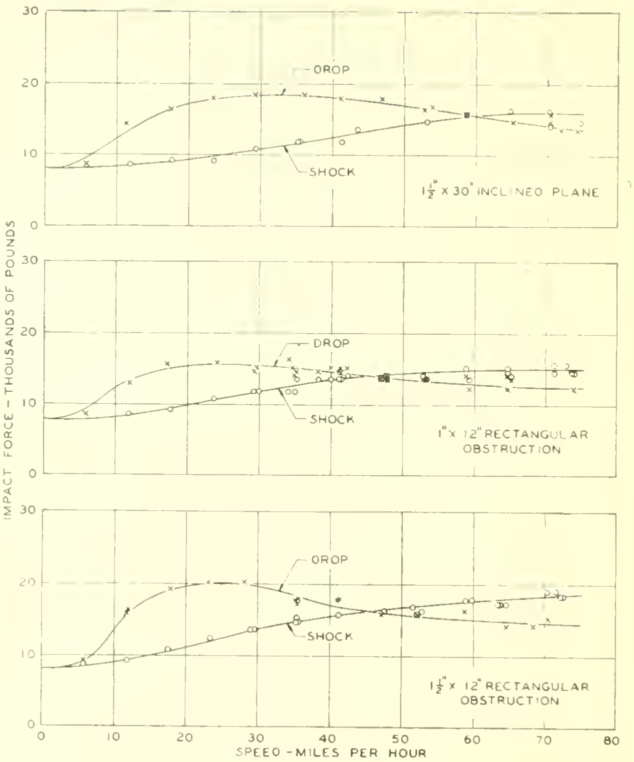


Figure 7.—Impact Tests with Artificial Obstructions and Single 12.75-24 Inch Balloon Tires at 90 Pounds Per Square Inch Inflation

These data show the distinct tendency for the curves representing shock reactions to flatten out at the higher speeds. It is significant that the shock reactions do not, even at a speed of 75 miles per hour, reach magnitudes as great as those reached by the corresponding drop reactions at speeds between 20 and 40 miles per hour. This relation leads to the important conclusion that, in so far as artificial obstructions such as these are concerned, the reactions of a motor vehicle wheel equipped with balloon tires tend to reach a maximum value at a speed between 20 and 40 miles per hour.

In connection with this discussion of the tendency for shock reactions to approach a maximum value as the speed is increased, it is noted that the maximum shock is reached with the 1-inch rectangular obstruction at a lower speed than is the case with the 1½-inch obstruction. This is in accord with the discussion of the theory underlying this phenomenon as brought out in the April, 1931, issue of Public Roads. When an obstruction is encountered by a tire in its travel over a road surface there is a tendency for the tire to envelop the obstruction by a local deformation of the cushioning material. As the speed increases the time during which a given obstruction is passing under the tire is decreased. If the obstruction is not too large and the tire has sufficient cushioning qualities, then at some speed the tire will substantially absorb the roughness condition without appreciably elevating the vehicle axle. The curve representing the shock reaction will increase up to that critical speed, and any increase in speed beyond the critical speed will show little or no increase in shock reaction for the given obstruction and tire equipment conditions. On the other hand, when the axle is raised as the result of a shock reaction, there is a corresponding and ensuing drop reaction. If the elevation of the axle due to shock gradually decreases with increases of speed, then the corresponding drop reaction will proportionately decrease.

In theory this elevation of the axle will approach zero as a limit when the speed is indefinitely increased. If this theoretical limit were ever reached the corresponding drop reaction would necessarily be zero. However, within the practical working limits defined by these tests, it appears that when the shock reaction approaches a maximum for a given obstruction and tire the elevation of the axle due to shock approaches a constant value as evidenced by the approach of the corresponding drop reaction to a constant value of relatively low magnitude.

Figures 6 and 7 bring out two other interesting facts. One is that an inclined plane of the dimensions of that used in the tests can produce appreciable shock reactions at high speeds. The other supports a conclusion drawn in earlier reports that the impact reactions developed by dual tires are slightly greater than those developed by single tires of the same type and having the same capacity as the duals.

TESTS INVOLVING NATURAL OBSTRUCTIONS

The natural obstructions used in the second phase of the program represent the worst which could be found at places where reasonable speeds could be attained in an area of more than 1,100 square miles, involving about 400 linear miles of primary and secondary roads in the vicinity of Washington, D. C. Tests were conducted at 28 locations throughout the full range of safe speeds possible at each. The load condition was the same as in the tests involving artificial obstructions. The dual 9.75—24 inch tires were used in tests at all locations, and in one case (fig. 22) additional tests were made with the single 12.75-24 inch tire.

As some of the tests over the natural obstructions were made at speeds considerably in excess of the legal limit, a special police detail of trained officers was made available to direct and control traffic in order that the

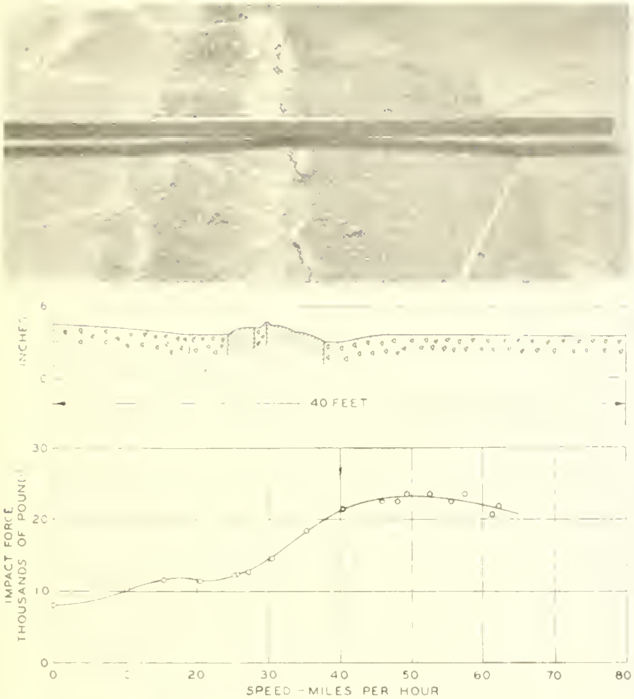


FIGURE 8.—IMPACT REACTIONS PRODUCED ON A CONCRETE PAVEMENT AT A HEAVED JOINT

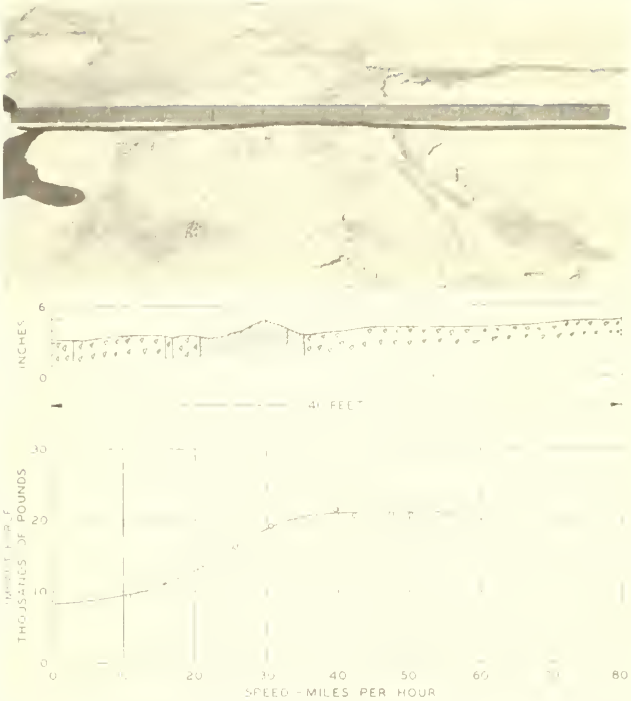


FIGURE 10.—IMPACT REACTIONS PRODUCED AT A ROUGH BITUMINOUS PATCH ON A CONCRETE PAVEMENT

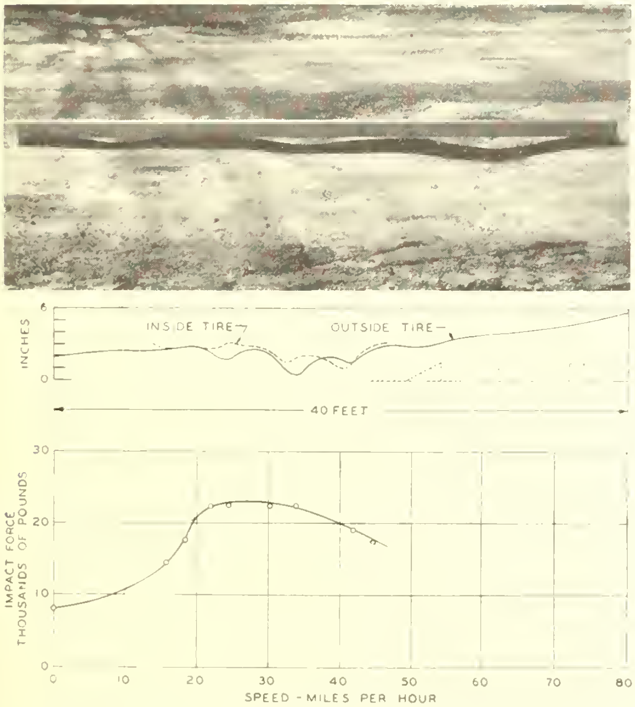


FIGURE 9.—IMPACT REACTIONS PRODUCED ON A WATER-BOUND MACADAM ROAD BY CORRUGATED SURFACE

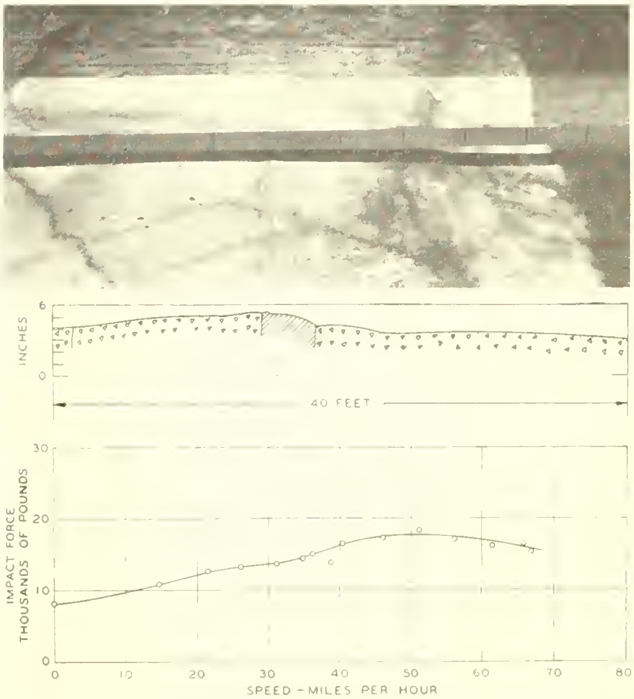


FIGURE 11.—IMPACT REACTIONS PRODUCED ON A CONCRETE ROAD BY A HEAVE AT A JOINT PARTLY REPAIRED WITH BITUMINOUS MATERIAL

high speeds might be safely attained. This courtesy was extended through the cooperation of the State roads commission and the commissioner of motor vehicles of the State of Maryland.

The data obtained in tests at 16 of the natural obstructions have been selected as representative and are presented in Figures 8 to 23, inclusive. These figures show the relation between the magnitude of the maximum impact reaction (whether shock or drop) and the vehicle speed as developed by the test bus at each of these very rough places in the highway surfaces.

The character of the surface at each location is shown by both a photograph and a profile taken along the path traversed by the right rear wheel of the test vehicle, which passed from left to right along the road surface as shown. The photograph gives a general

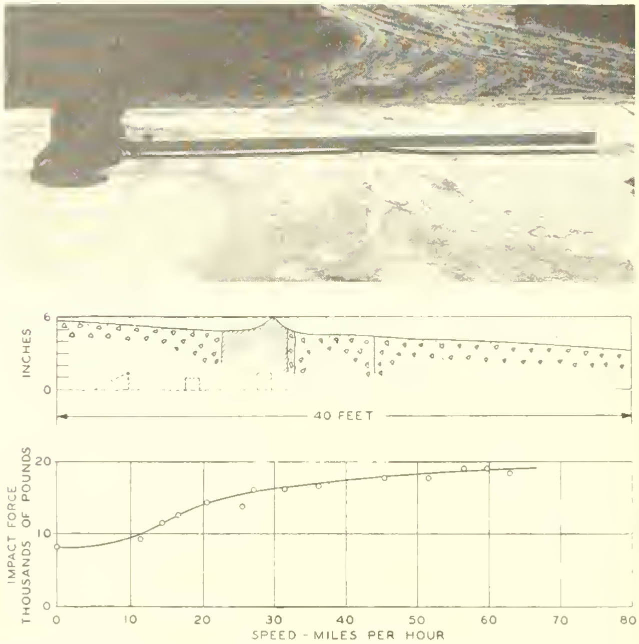


Figure 12.—Impact Reactions Produced at a Heaved Joint on a Concrete Pavement

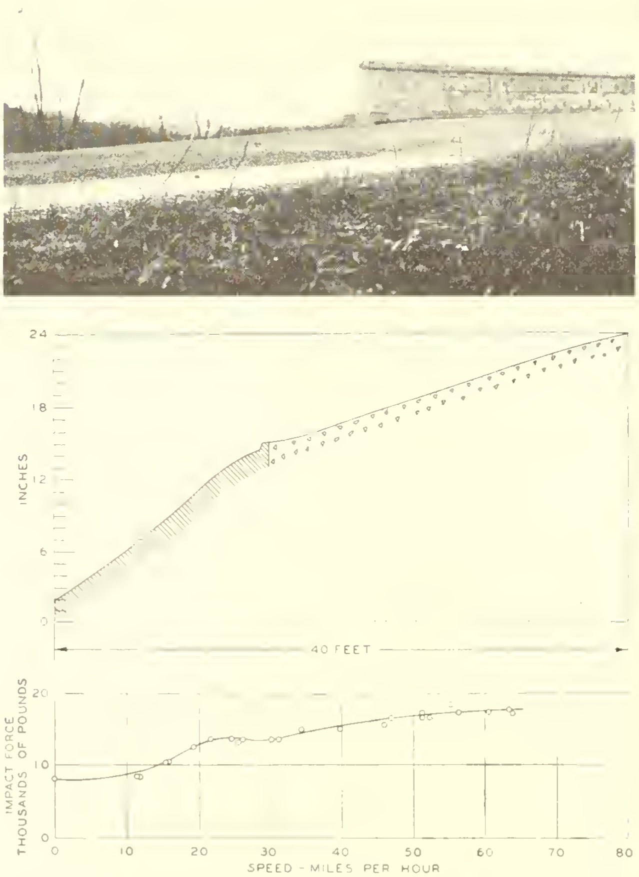


Figure 13.—Impact Reactions Produced on a Concrete Bridge by the Sharp Rise to the Bridge from a Road at a Lower Grade

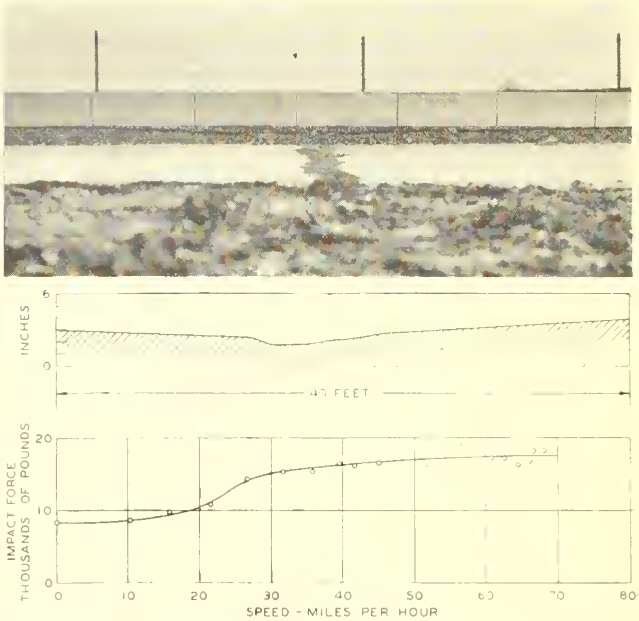


Figure 14.—Impact Reactions Produced on a Bituminous Pavement by Settlement of Back-fill Over a Culvert Pipe

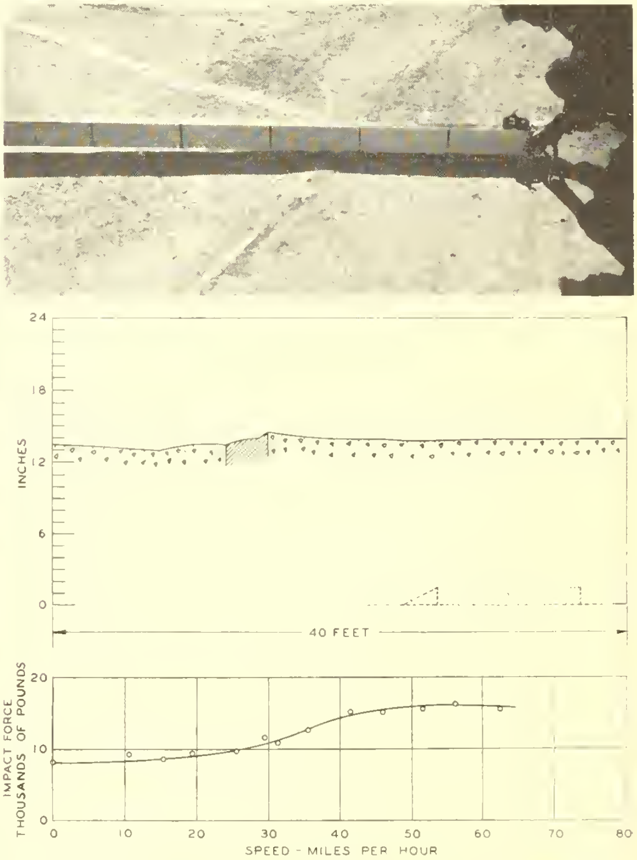


Figure 15.—Impact Reactions Produced on a New Concrete Bridge by Settlement of Fill Under Concrete Approach

picture of the rough spot. The straightedge, to be seen in most of the photographs, is 4 inches in height and 12 feet long, and is marked off in 1-foot intervals throughout its length. The shadow thrown on the pavement by this straightedge gives an excellent idea of the roughness of the road surface. The profiles, shown to a greatly distorted scale, are intended to give the details of the surface for engineering purposes rather than a "picture" of conditions as they existed. This fact should be kept in mind whenever an attempt is made to compare the size or shape of the natural rough-

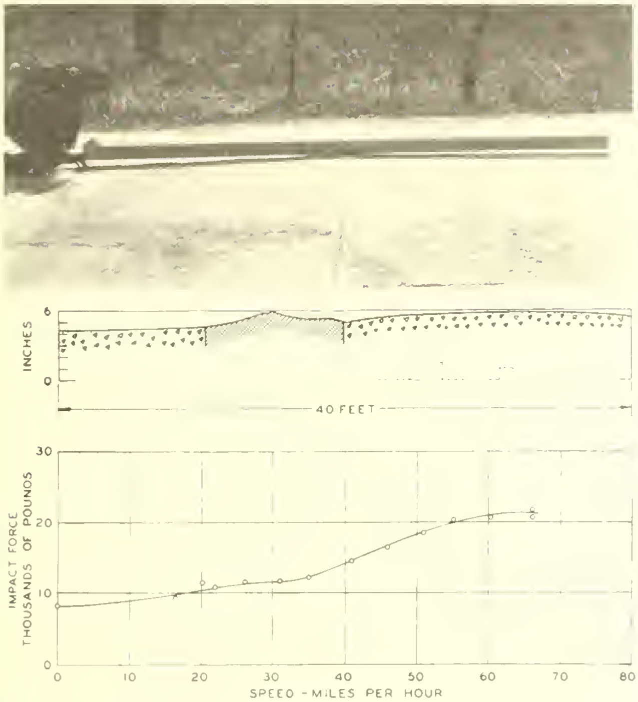


Figure 16.—Impact Reactions Produced on a Concrete Road. A Heaved Joint has been Repaired with Bituminous Mixture, but the Heaving has Apparently Continued

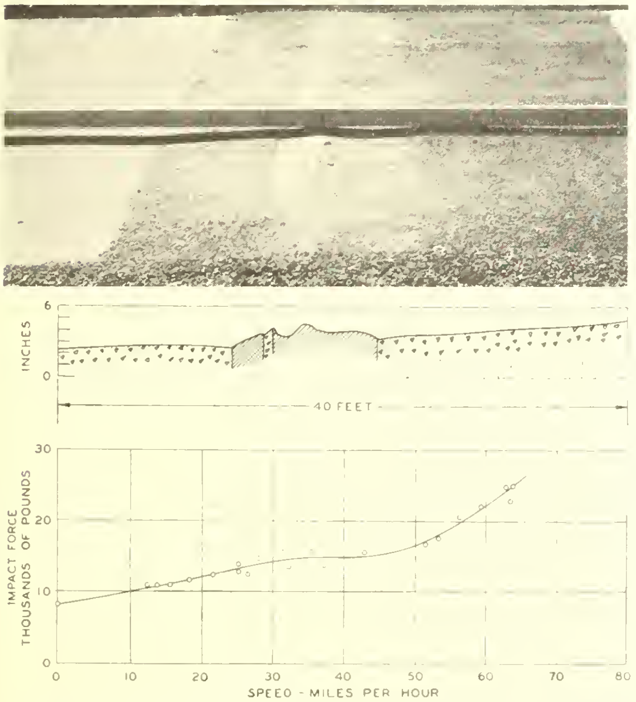


Figure 17.—Impact Reactions Produced on a Concrete Pavement at a Bad Joint which has been Patched with Bituminous Mixture

nesses as shown on the profiles with that of the artificial obstructions used in the first part of the program of tests. These three artificial obstructions, the 1½ by 30 inch inclined plane, the 1½ by 12 inch rectangular obstruction, and the 1 by 12 inch rectangular obstruction, are shown on each drawing to the same distorted scale as the profiles in order that their shapes may be compared with those of the natural obstructions.

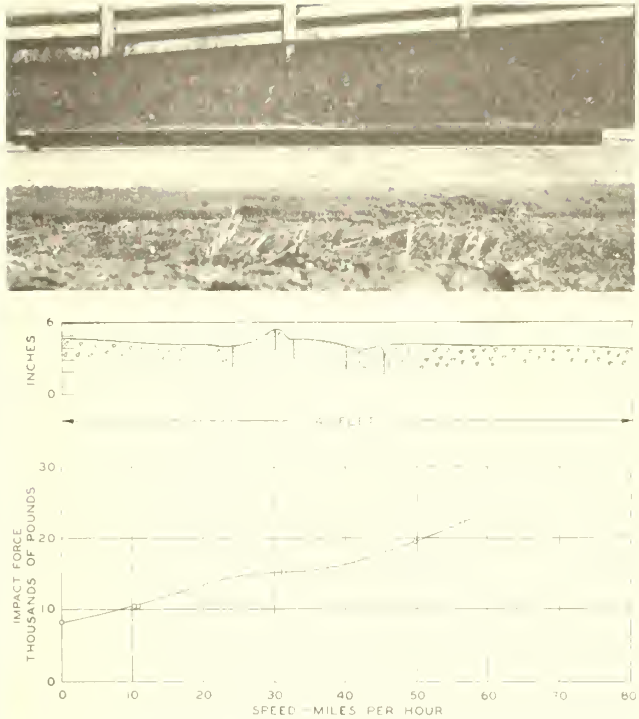


Figure 18.—Impact Reactions Produced at a Bituminous Patch in a Concrete Pavement

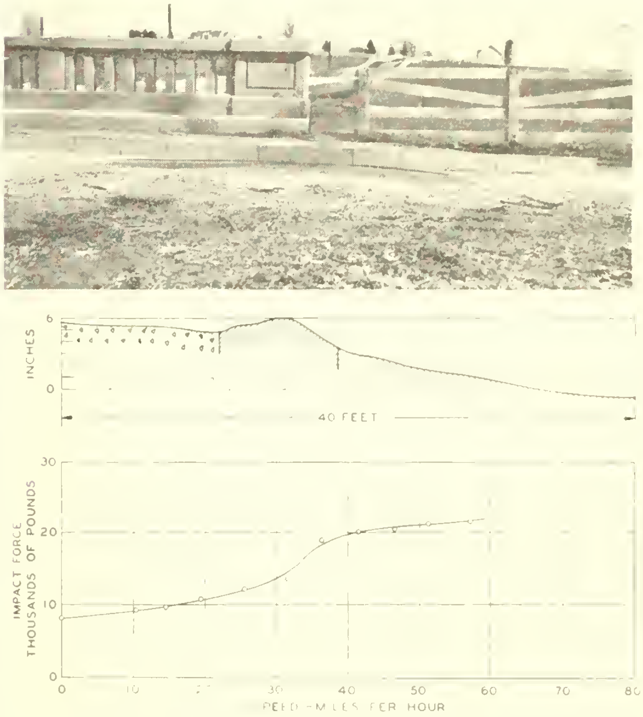


Figure 19.—Impact Reactions Produced on a Macadam Road by the Unevenness of the Grade at a Bridge Connection

The impact force produced at a given natural obstruction may not be directly comparable to that developed in the tests with the artificial obstructions, even though the two appear to be of approximately corresponding dimensions. The shape of the artificial obstructions is conventional, and in the tests they were placed on an extremely smooth, level stretch of new concrete pavement which had a very flat crown. Dur-

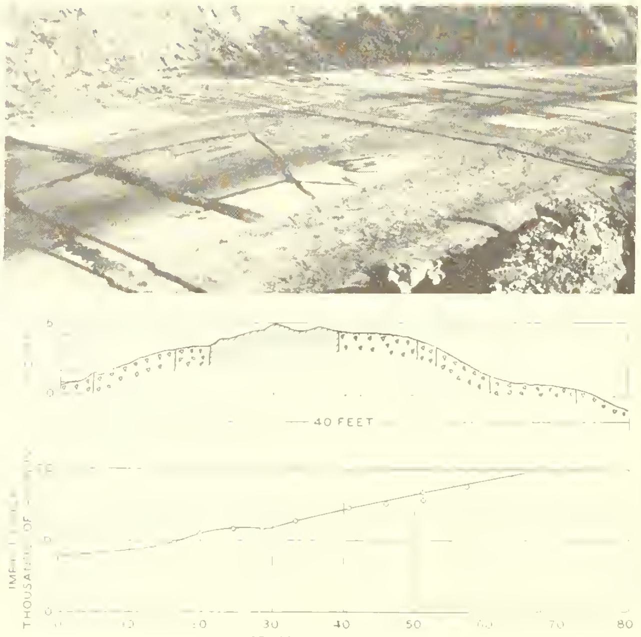


FIGURE 20.—IMPACT REACTIONS PRODUCED ON A CONCRETE ROAD AT A SHARP VERTICAL CURVE AT THE TOP OF A HILL

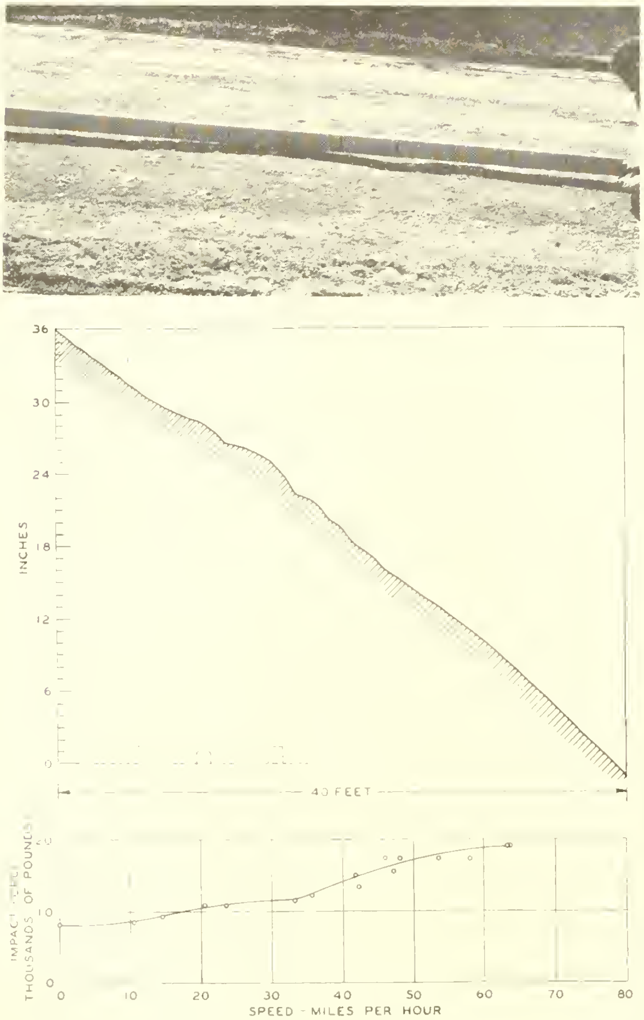


FIGURE 21.—IMPACT REACTIONS PRODUCED ON AN OILED GRAVEL ROAD BY CORRUGATIONS ON A STEEP GRADE

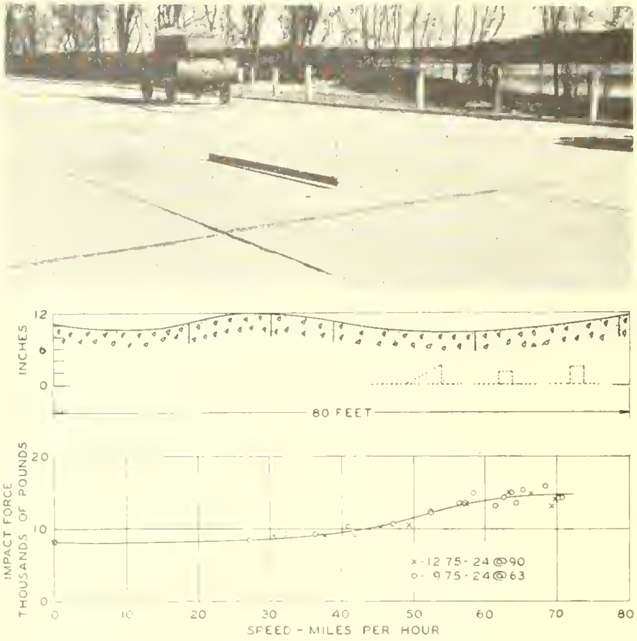


FIGURE 22.—IMPACT REACTIONS CAUSED BY THE UNEVEN SETTLEMENT OF FILL UNDER A NEW CONCRETE PAVEMENT. THE MASONRY UNDERPASS OF A GRADE SEPARATION LIES DIRECTLY UNDER THE HUMP IN THE GRADE

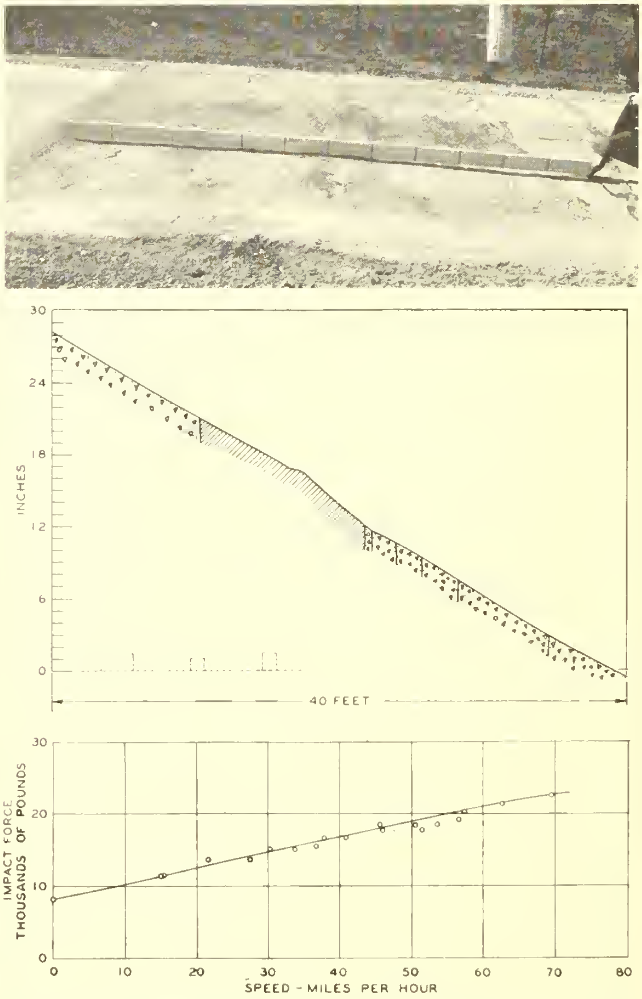


FIGURE 23.—IMPACT REACTIONS PRODUCED ON A HILL WHERE A CONCRETE PAVEMENT HAS BEEN EXTENSIVELY PATCHED WITH BITUMINOUS MIXTURE

ing the test the right rear wheel approached the obstruction along this smooth surface and at the instant of contact the body of the vehicle and the rear springs were in what may be termed the normal rolling position. Even during the impact at the right rear wheel, the other three wheels were passing over smooth pavement. Under such conditions the influence of the road surface adjacent to the artificial obstructions may be regarded as a minimum and probably negligible. In the case of the tests with the natural obstructions, however, such is not the case. The obstructions themselves conformed to no conventional design, the adjacent pavement was more or less uneven, the crown was not always flat nor the grade level. The profile of the road surface was even different under the two tires of the right rear wheel, an extreme case of this being shown in Figure 9. The result of these complicating factors was that the four wheels of the vehicle approached the natural obstruction over a surface that was not smooth and at the instant of contact of the right rear wheel with the rough place in the road surface the position of the vehicle body and the state of compression of the rear springs was being influenced by what had occurred and was occurring at all of the four wheels of the vehicle. It is, therefore, to be expected that the conditions attending the tests with the natural obstructions might combine to exert a considerable influence on the relation between the magnitude of the impact reaction and the speed of the vehicle.

Early in the tests involving natural obstructions it was indicated that the segregation and identification of shock and drop types of reaction would necessitate an undue length of time at each location. As the magnitude of the maximum reaction at a large number of rough spots was of greater significance, the force measured for each test run is the maximum developed, whether shock or drop in type. However, the shape of the force versus speed curves is, in many instances, characteristic of the drop type of reaction at the lower speed range and characteristic of the shock type at the higher speeds.

The data from the tests over the numerous natural obstructions evidence considerable variety in the shape of the curves showing impact force as influenced by vehicle speed. Because of the infinite variety of the roughness conditions which can and do occur on the highways, the results of tests involving only one rough spot or type of roughness condition should not be taken as representative of the reactions which may occur. So far as the tests involving these 28 natural obstructions may be considered to be representative, it is indicated that the maximum reactions developed by balloon tires on the highway are of the same order as the maximum reactions developed in the tests with artificial obstructions, or about three times the static wheel load. These tests also show that the maximum reaction on the road may occur at nearly any operating speed, depending upon the particular attributes of the surface roughness involved. As the roughness conditions on the road possess individual characteristics, each one unique with respect not only to the local roughness but to the general profile of the adjacent road surface as well, so do the relations between impact force and vehicle speed show corresponding individuality.

The conclusion, based upon results of tests with artificial obstructions on an otherwise smooth and level road, that the reactions may reach maximum values between 20 and 40 miles per hour, must be somewhat modified when the results of tests involving "natural" rough spots or obstructions are considered. In a few cases the reactions caused by the natural obstructions which were used do actually reach maximum values at a speed of 40 miles per hour or less. The preponderant tendency, however, is for the reactions to show a general increase with increases in speed.

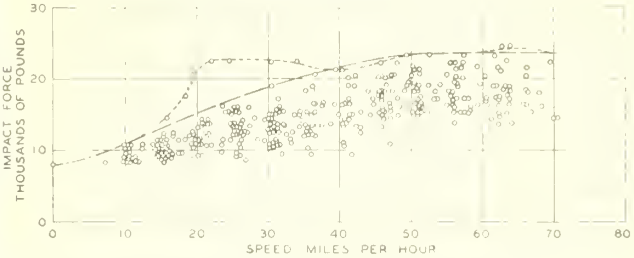


Figure 24.—Composite of All Data Derived from Tests Involving Severe Natural Obstructions

A better conception of the real significance of the reactions which may and do occur as a result of rough spots in the pavement surface may be obtained by referring to Figure 24, in which the data from all tests at the 28 locations have been plotted in a common graph. In this figure the dotted line represents the envelope of all the force versus speed curves for the individual conditions. The broken line, however, more nearly represents the maximum reactions to be expected throughout the speed range covered by this large group of data for natural roughness conditions. Between the limits of 15 and 30 miles per hour the dotted-line curve is based solely on points plotted from data which represent a single natural roughness condition. (See fig. 9.) The general trend, as indicated by the broken line, is for the reaction to increase up to a speed of about 50 miles per hour, when a general maximum is reached which is constant up to 70 miles per hour. The reactions, in percentage of static load, indicated by the broken line as maxima to be expected as a result of these natural obstruction tests, are given in Table 2.

Table 2.—*General maximum reactions produced by 28 natural rough spots on highways*

Speed	Impact reaction	Ratio of impact reaction to static load	Speed	Impact reaction	Ratio of impact reaction to static load
Miles per hour	*Pounds*	*Per cent*	*Miles per hour*	*Pounds*	*Per cent*
0	8,000	100	40	21,800	273
10	11,000	138	50	23,500	294
20	15,200	190	60	23,700	296
30	19,000	238	70	23,700	296

From this it may be concluded that the maximum reaction which a heavy vehicle equipped with balloon tires may be expected to produce when traveling over a large number of natural rough spots is about three times the magnitude of the static wheel load, and that at 40 miles per hour reactions of about two and three-quarters times the static wheel load or 92 per cent of the maximum may be expected to occur.

MAGNITUDE OF REACTION VERSUS FREQUENCY

The tests concerning the magnitude of impact reaction developed versus the frequency with which such forces occur on actual highways were made on 14 typical roads. Seven representative surface roughness conditions for each of the concrete and bituminous types of construction were selected in the same area in the vicinity of Washington in which the natural obstructions were found. The surface roughness of the seven roads in each group ranged from the best to the worst which could be found in the area examined. The length of each test section was one-half mile and none of the natural obstructions which were used in the tests previously described, or any other exceptional condition, was included in the representative test sections. Tests on each road were made at speeds of 20, 30, and 40 miles per hour.

These roads were selected in the following manner: Within the area surveyed a number of sections were considered to be of sufficient length and carrying such volume of traffic as to warrant consideration of them as representative surfaces. These sections were then examined with a device for indicating relative road surface roughness. This roughness indicator was described in Public Roads, volume 7, No. 7, September, 1926. Although this instrument can not be said to measure the actual roughness of a road surface, it serves as a valuable guide in determining the relative roughness of road surfaces and it was so used in this investigation. Readings were taken at $\frac{1}{10}$-mile intervals over about 5 miles on each section, and from the data thus obtained a $\frac{1}{2}$-mile length was finally selected as uniform and representative of the section. It is interesting to note that the impact-frequency curves for the seven roads of each type arranged themselves, with one minor exception, in the same order as the road surfaces were classified with the relative roughness indicator.

The impact frequency tests were made at several speeds to avoid the arbitrary selection of any one speed in conducting the tests. The maximum speed was 40

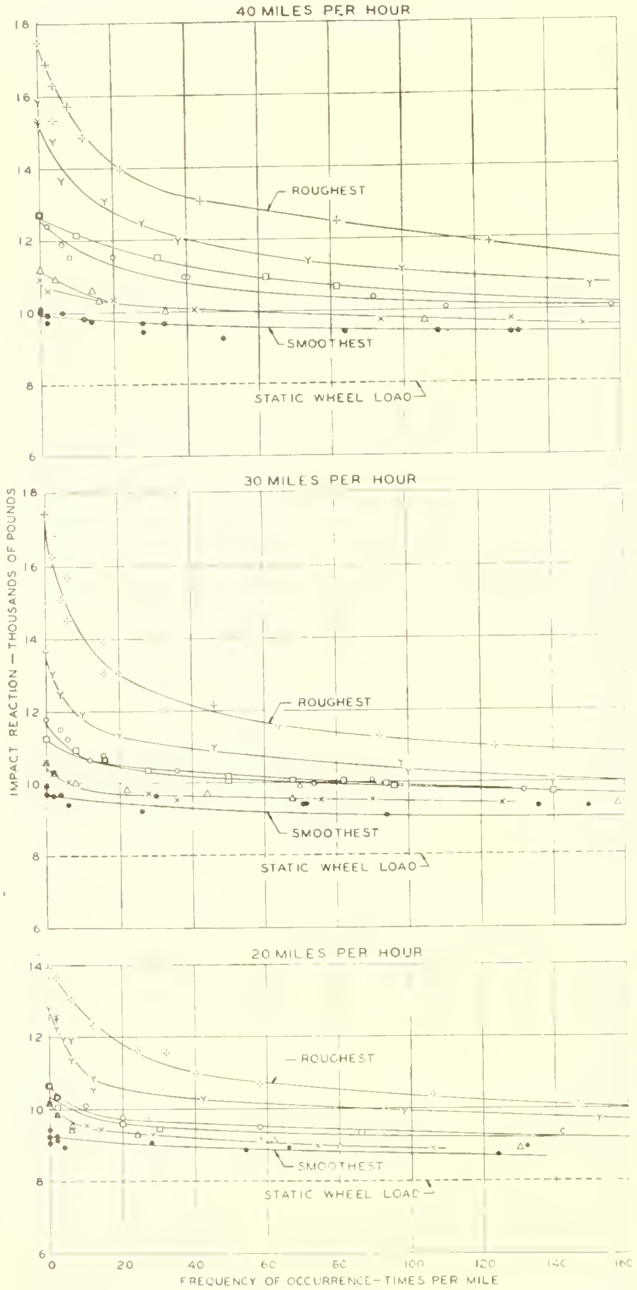


Figure 25.—Curves of Magnitude of Impact Reaction Versus Frequency of Occurrence for Concrete Road Surfaces

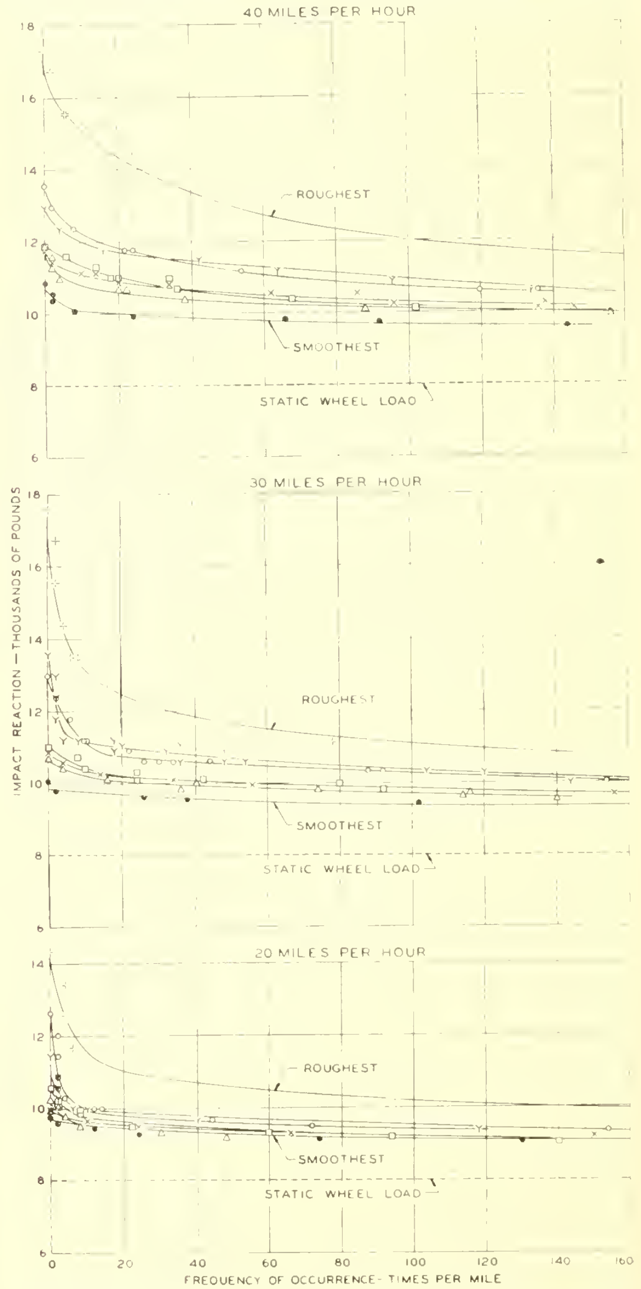
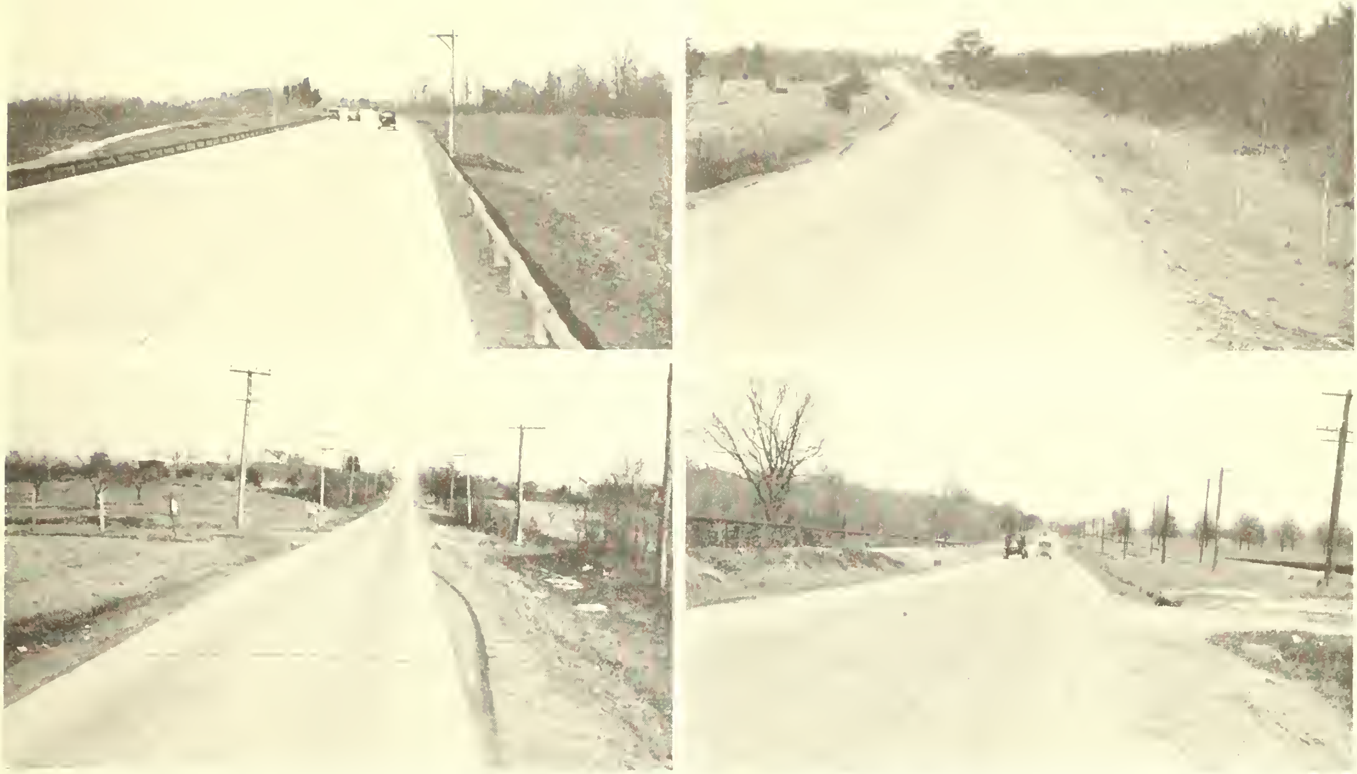


Figure 26.—Curves of Magnitude of Impact Reaction Versus Frequency of Occurrence for Bituminous Road Surfaces



Upper.—The Smoothest Concrete Surface Tested
Lower.—The Roughest Concrete Surface Tested

Upper.—The Smoothest Bituminous Surface Tested
Lower.—The Roughest Bituminous Surface Tested

Figure 27.—Examples of Roads on Which Impact-Frequency Tests Were Made

miles per hour, because that was the maximum which could be maintained conveniently on all of the half-mile test sections with safety and it was within legal speed limits. Higher speeds were not used because too much time would have been wasted in awaiting favorable traffic conditions to make the entire test run at constant speed without weaving in and out of traffic. In the previous tests involving artificial and natural obstructions it was found that maximum reactions were reached or reasonably approached at a speed of 40 miles per hour. Furthermore, tests conducted at this speed are representative of a considerable volume of normal traffic.

The force versus frequency data are presented in Figures 25 and 26. There is very little difference between the ranges of impact-producing roughness of the concrete and bituminous pavements tested. The smoothest surfaces and the roughest surfaces of each pavement type tested appear to be, respectively, of comparable orders. In Figure 27 photographs of the best and worst sections of each type are shown.

In Figure 28 the data are rearranged to give a more generalized picture of the range in forces produced as influenced by the frequency of occurrence and the vehicle speed. In this figure curves have been drawn representing the forces, expressed in terms of static wheel load, produced by the best and the worst surface conditions of the roads represented in the tests.

These force versus frequency data indicate that the effect of speed is of comparatively little significance when the road surface is reasonably smooth. Even on the smoothest surfaces reactions of from 110 to 125 per cent of the static wheel load occur with sufficient frequency to constitute a factor which should be considered in pavement design. As the surface roughness increases the frequency at which a given impact force

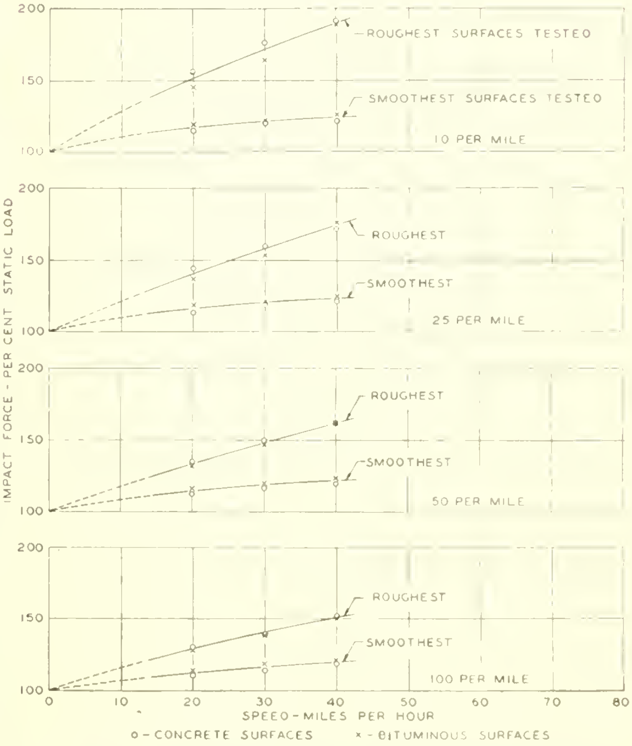


Figure 28.—The Influence of Road Surface Condition on the Impact Force-Frequency Relation

(Continued on p. 151)

EXTRA WIDE HIGHWAYS ON STATE SYSTEMS

THE accompanying table shows the extent to which the construction of State roads having more than two traffic lanes had progressed by the end of the calendar year 1931. A total of 3,790 miles was reported, of which 2,230 miles were of 3-lane width, 1,385 of 4-lane, 83 of 5-lane, and 92 of 6 lanes and over.

It should be noted these figures refer to rural highways on the State systems only. There is also a very great mileage of city streets and parkways having more than two lanes, including many broad traffic arteries which connect with the State systems at the city limits. No figures are available on the extent to which the counties have participated in the development of extra wide surfaces.

Wilkinson Boulevard, Between Gastonia and Charlotte, N. C.

The recently completed Mount Vernon Memorial Highway, 15 miles long, with a minimum width of 40 feet, is under the jurisdiction of the Federal Government, and is therefore not included in the mileage reported for Virginia.

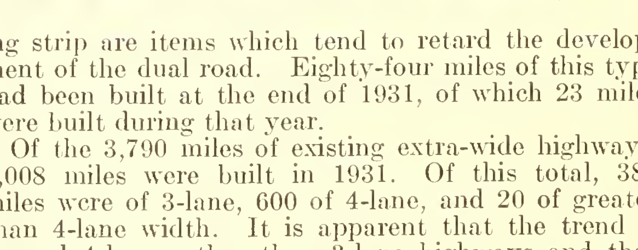
The excess of 3-lane over 4-lane roads is very largely accounted for by the figures from the State of New York, which reports 914 miles of 3-lane road and only 185 miles having 4 lanes. Illinois, in contrast, has only 7 miles of 3-lane roads and 251 miles with 4 lanes.

The results of a cooperative study of highway traffic capacity were reported by A. N. Johnson, dean, College of Engineering, University of Maryland, in Public Roads, volume 13, No. 3, May, 1932. In this report it is stated that the working capacity of a 3-lane road, defined as the point at which congestion first becomes apparent, was found to be about 2,000 vehicles per hour, an increase of approximately 100 per cent over that of a 2-lane road. The working capacity of a 4-lane road was found to be about 3,000 vehicles per hour. Considerations of safety may influence the choice of a 4-lane width, as it eliminates the conflict between vehicles on the single passing lane, which may be an appreciable hazard with heavy traffic on a 3-lane road.

Only 175 miles of roads having more than four lanes have been built on the State highway systems. While such broad boulevards are not uncommon in large cities, there are few sections of State highways on which the traffic justifies the construction of more than four lanes. Even when there is sufficient volume, as in the case of a road connecting two great cities, the problem of distributing traffic at the urban terminals of a 6 or 8 lane highway is very difficult. It is very often preferable to open up an alternative route having four lanes or less.

40-Foot Brick Pavement North of Akron, Ohio

The dual road, in which the opposing traffic streams are separated by parkway or car tracks, has distinct advantages from the standpoint of safety, in addition to the opportunities for landscape treatment afforded by a central parkway. The increased width of right of way required and the cost of improving the separating strip are items which tend to retard the development of the dual road. Eighty-four miles of this type had been built at the end of 1931, of which 23 miles were built during that year.



Dual Highway Between Detroit and Ann Arbor, Mich.

Of the 3,790 miles of existing extra-wide highways, 1,008 miles were built in 1931. Of this total, 388 miles were of 3-lane, 600 of 4-lane, and 20 of greater than 4-lane width. It is apparent that the trend is toward 4-lane rather than 3-lane highways and that designs involving more than four lanes are not finding much favor at the present time.

MILEAGE OF EXTRA WIDE SURFACED ROADS ON STATE HIGHWAY SYSTEMS, 1931

[Includes only rural roads over two lanes wide, as reported by State authorities]

State	Total mileage of extra wide roads	Mileage classified by number of lanes				Mileage with contiguous lanes all of one type of surface																	Mileage with contiguous lanes of two types of surface											Dual roads—Lanes separated by parkway or trackway [2]		
						Total	Bituminous macadam and surface treatments [1]				Bituminous concrete				Portland cement concrete				Brick or other block pavement				Total	Bituminous macadam and cement concrete				Bituminous concrete and cement concrete				Brick or block and cement concrete		Number of lanes		
							Number of lanes				Number of lanes				Number of lanes				Number of lanes					Number of lanes				Number of lanes				Number of lanes				
		3	4	5	6 and over		3	4	5	6 and over	3	4	5	6 and over	3	4	5	6 and over	3	4	5	6 and over		3	4	5	6 and over	3	4	5	6 and over	3	4	4	6	8
Alabama	1.3		1.3			0.5										0.5																		[3] 0.8		
Arizona	None.																																			
Arkansas	3.7		3.7			.4										0.4							3.3		[4] 3.3											
California	314.3	184.6	77.2	13.0	[5] 39.5	226.4	[6] 36.8	[7] 26.9	5.9		20.8	3.8	5.4	[8] 6.2	66.5	32.5	1.0	[9] 20.6					87.9	4.8			[10] 1.0	55.7	14.0	0.7	[11] 11.7					
Colorado	1.9		1.9			1.9										1.9																				
Connecticut	39.6	1.7	36.0	1.9		39.2		2.5			1.5	2.0			0.2	30.9	1.9			0.2			.4		.4											
Delaware	24.7		24.7			3.7										3.2				.5			6.0						1.9				4.1	[12] 15.0		
Florida	38.2	37.0	.2	1.0		38.2	[13] 14.3		[13] 1.0		4.5				15.9				[14] 2.3	.2																
Georgia	28.8	17.2	11.6			24.1	2.6				1.2	6.8			10.1	3.4							4.7	[15] 3.3	[15] 1.4											
Idaho	None.																																			
Illinois	258.8	7.3	251.0	.5		258.8					.2				7.1	250.9	.5			.1																
Indiana	110.5	40.9	64.6	3.3	1.7	76.9	.2	1.4			3.1	.9	.3		11.2	48.6	.6	1.3	3.1	3.5	2.4	[16] 0.3	33.6	21.4	9.1			.8	[17] [illegible]		.1	[illegible]	[illegible]			
Iowa	None.																																			
Kansas	None.																																			
Kentucky	None.																																			
Louisiana	13.4		13.4			13.4										13.4																				
Maine	36.8	36.8				34.3	.7								33.6								2.5	2.5												
Maryland	47.1	7.0	39.6		.5	14.1						3.8			.2	10.1							30.4		.7			6.8	22.9					[18] 2.1		[19] 0.5
Massachusetts	224.2	148.1	74.3		1.8	162.3	82.4	15.3			6.8	.7			30.2	25.5			[20] 1.4				36.0	27.3	8.0						[21] .7			[22] 24.8	[23] 1.1	
Michigan	205.9	6.7	172.8		26.4	176.8		.5			1.4	2.5			5.3	166.1		1.0					3.7						3.7							[25] 25.4
Minnesota	156.7	84.0	72.7			156.7					12.2	14.9			71.8	46.4				[24] 11.4																
Mississippi	.9		.9																															.9		
Missouri	30.5		27.9		2.6	30.5										27.9		2.6																		
Montana	None.																																			
Nebraska	None.																																			
Nevada	2.9	2.0		.9		2.9	[26] 1.0		[26] .9						1.0																					
New Hampshire	9.5	9.5				9.5					3.7				5.8																					
New Jersey	332.5	236.9	82.7	3.7	9.2	327.7	17.3	11.9	1.7	4.6	218.1	59.4	2.0	3.6	.9	1.3			[20] .6	[20] 5.3		[20] 1.0												[27] 4.8		
New Mexico	None.																																			
New York	1,104.0	913.8	184.9		5.3	1,014.2									827.1	181.8		5.3					89.8	[28] 66.2				17.8	3.1			2.7				
North Carolina	45.3	31.3	14.0			45.3					25.3				6.0	14.0																				
North Dakota [29]	2.6	.8	1.8			2.6									.8	1.8																				
Ohio	176.7	101.8	23.7	51.2		173.0	14.9	.3	3.0		13.5	4.4	2.5		51.8	1.6	11.3		21.6	13.7	34.4		3.7						3.7							
Oklahoma	3.5	3.5				3.5									3.5																					
Oregon	None.																																			
Pennsylvania	82.5	59.9	17.1	3.0	2.5	57.1		.1		[13] 1.7	.8	.9			38.2	11.3	1.3	.8	1.5	.4	.1		25.4	19.4	4.4	1.6										
Rhode Island	58.3	3.7	54.6			58.3						15.1			3.7	39.5																				
South Carolina	16.2	7.9	6.3	1.5	.5	16.2					7.1	4.6	1.5	.3	.8	1.7		.2																		
South Dakota [30]	None.																																			
Tennessee	33.1	3.3	29.7	.1		32.7		4.9			.5				2.4	24.8	.1						.4					.4								
Texas	15.3	1.1	12.6		1.6	15.3						4.3			1.1	8.3						1.6														
Utah	4.7		4.7			1.4										1.4							3.3						3.3							
Vermont	None.																																			
Virginia	128.2	119.2	9.0			119.2	55.2				20.8				43.2																			[31] 9.0		
Washington	40.5	13.7	26.5		.3	35.8									9.3	26.5							4.7	4.4												
West Virginia	20.1	10.1	6.7	3.3		18.4					.7		.8		6.3	1.3	1.2		3.1	3.7	1.3		1.7										1.7			
Wisconsin	176.5	140.2	36.3			176.5	[32] 116.2	[15] 11.6			.7				23.3	24.7																				
Wyoming	.6		.6			.6										.6																				
Subtotal		2,230.0	1,385.0	83.4	91.9		341.6	75.4	12.3	6.3	312.9	124.1	12.7	10.1	1,277.3	1,002.3	17.9	31.8	33.6	39.0	38.2	2.9		149.3	27.3	1.6	1.3	81.5	52.9	.7	12.5	3.8	6.6	57.4	1.1	25.9
Total	3,790.3					3,368.4	435.6				489.8				2,329.3				113.7				337.5	179.5				147.6				10.4		84.4		

---

[1] Bituminous macadam unless otherwise noted.

[2] These columns give mileage of roads with parkway, or railway track (not usable by motor vehicles) in center between more than 2 lanes of various types as noted.

[3] Car tracks on a 10-foot wide center strip, with 2 lanes of cement concrete on each side.

[4] Consists of treated gravel between concrete lanes.

[5] Includes 10.5 miles over 6 lanes wide.

[6] Includes 20.6 miles of treated gravel

[7] Includes 24.0 miles of treated gravel

[8] Includes 4.8 miles over 6 lanes wide.

[9] Includes 2.8 miles over 6 lanes wide

[10] Includes 0.7 miles over 6 lanes wide.

[11] Includes 2.2 miles over 6 lanes wide.

[12] Consists of a 50-foot central grass parkway with 2 lanes of cement concrete on each side.

[13] Consists of treated water-bound macadam.

[14] Includes 1.8 miles of asphalt block.

[15] Consists of bituminous macadam and bituminous concrete.

[16] Over 6 lanes wide.

[17] Consists of bituminous concrete and brick.

[18] Consists of 2 lanes of bituminous macadam and 2 lanes of cement concrete separated by tracks.

[19] Consists of 4 lanes of bituminous macadam and 4 lanes of cement concrete separated by parkway

[20] Consists of stone block pavement.

[21] 2 center lanes of bituminous concrete include tracks.

[22] Grass plot reservation in center, with lanes of cement concrete on each side.

[23] Consists of 4 lanes of cement concrete on each side of center parkway with track

[24] Includes 7.8 miles of asphalt block.

[25] Consists of a 23-foot parkway in center and 2 lanes of cement concrete on each side.

[26] Consists of bituminous-treated gravel.

[27] Consists of 20 feet of bituminous concrete and 20 feet of cement concrete separated by a 20 foot gravel strip

[28] Includes 2.2 miles of treated water-bound macadam and cement concrete.

[29] Excludes 69.3 miles of gravel roads 3 to 6 lanes wide reported as largely through cities and towns

[30] Excludes 40.8 miles of 3 lane gravel roads reported as plain and treated.

[31] Consists of cement concrete lanes with railway tracks in central private way

[32] Includes 103.8 miles of treated water-bound macadam.

STATE AND LOCAL GOVERNMENTAL AGENCIES EXEMPT FROM FEDERAL EXCISE TAXES

State and local highway officials who have the duty of purchasing gasoline, lubricating oil, automobiles, tires, or other articles and commodities on which excise duties were levied under the revenue act of 1932 may be interested in the following extracts from the regulations of the Bureau of Internal Revenue, United States Treasury Department.

From Regulations 42, relating to the taxes on telegraph, telephone, radio, cable facilities; transportation of oil by pipe line; safe deposit boxes, checks, etc.; and electrical energy—

> ART. 19. *Services rendered to the United States or to any State or Territory or to the District of Columbia.*—Telephone, telegraph, cable and radio dispatches, messages, and conversations relating to Government business, which originate in the United States and which are a charge against the United States, the District of Columbia, a State, or Territory, and are paid from the funds thereof, are exempt from the tax. Messages, conversations, and dispatches which are not paid from such funds are not exempt from tax, even though they relate to Government business.
>
> The words "State" and "Territory" include political subdivisions thereof, such as counties, cities, towns, and other municipalities.
>
> ART. 36 (in part). * * * The checks, drafts, or orders drawn by officers of the United States or of a State, county, or municipality, or of a foreign government, in their official capacities, against public funds standing to their official credit and in furtherance of duties imposed upon them by law are not subject to the tax.
>
> ART. 41 (in part). * * * Electrical energy furnished to the United States or to any State or Territory, or political subdivison thereof, or the District of Columbia is exempt from tax.
>
> This exemption does not apply to payments for electrical energy for domestic or commercial consumption furnished by governmentally or municipally owned electrical power companies.
>
> The exempt agencies must establish their right to exemption by submitting the necessary evidence to the person furnishing the electrical energy.

From Regulations 44, relating to the taxes on lubricating oil; brewer's wort and malt products; grape products; matches; soft drinks; and gasoline—

> ART. 9. *Exempt sales to States and political subdivisions thereof.*—If articles are sold directly to a State or political subdivision thereof for use in the exercise of an essential governmental function, the tax does not attach, but sales to a dealer or distributor are taxable even though the manufacturer has knowledge that the articles are destined for ultimate use by or resale to a State or political subdivision thereof. Sales to the Government of the United States, the District of Columbia, or a Territory or possession of the United States are taxable.

From Regulations 46, which deal with excise taxes imposed under Title IV of the revenue act of 1932 on sales by the manufacturer, producer, or importer, of tires and inner tubes, toilet preparations, furs, jewelry, etc., automobiles, motor cycles, etc., radio receiving sets and phonograph records, mechanical refrigerators, sporting goods, firearms, shells, and cartridges, cameras, candy, and chewing gum—

> ART. 17 (in part). * * * The tax does not attach to sales of any articles to States or political subdivisions thereof to be used in the exercise of an essential governmental function, provided such sales are made direct by the manufacturer to a State or political subdivision thereof without any intervening sale to a dealer or distributor.

The claim for exemption from these excise taxes is made by the manufacturer or producer, who must furnish the Bureau of Internal Revenue with satisfactory evidence that the purchase was made with Government funds and for the official use of the Government agency concerned.

Articles or commodities purchased under the tax exemptions outlined above must not be resold or appropriated to personal use, as such action is in violation of the Federal statutes and is punishable under section 1114 of the revenue act of 1926, which, as stated in Regulations 46, article 73, provides that—

> any person who willfully fails to pay or collect any tax due, file return or keep records, or who attempts in any manner to evade or defeat the tax, is subject to a fine of $10,000 or imprisonment, or both, with costs of prosecution, and is also liable to a penalty equal to the amount of the tax not collected or paid.

VITRIFIED BRICK ON CONNECTICUT AVENUE EXPERIMENTAL ROAD

Experiment No. 6, the most northerly portion of the Connecticut Avenue experimental road, was constructed of vitrified brick on a foundation of 1:3:7 gravel concrete. The photographs on the back cover of this issue show the method of construction and the appearance of the surface shortly after completion. The front cover shows the condition of the pavement in the summer of 1932 after 19 years of service.

The entire surface was constructed during the spring of 1913. On the concrete foundation a cushion of sand was spread, rolled with a 300-pound roller, and struck off to a true depth of 2 inches. The brick were laid in straight courses at right angles to the curb, and the surface was rolled with a 5-ton tandem roller until they were firmly bedded in the sand cushion. The joints were filled with two applications of 1:1 Portland cement grout, the first application being swept into the joints and the second squeegeed over the pavement. Immediately after completion of the grouting sand was spread over the pavement to a depth of one-half inch and kept wet for seven days. This procedure differed from modern methods in the thickness of the sand cushion and the use of cement rather than bituminous grout in the joints. In brick pavements to-day the sand cushion is seldom more than 1 inch thick.

Longitudinal joints were provided along each curb, but no transverse expansion joints were constructed and the ends of the pavement were practically fixed.

The experiment was constructed with brick obtained from many sources and of widely differing characteristics. They were laid in 14 subsections, A to N. The cost of the surface, including the concrete base, was $2.58 per square yard.

Sections K, L, M, and part of N were constructed over a heavy fill which some years ago settled to a considerable extent, causing failures in the overlying pavement and necessitating heavy repairs during 1919, 1920, and 1922. The fill has continued to settle and maintenance costs on these sections have been high. Such costs are not properly chargeable to surface maintenance and have not been included in the computations of maintenance charges.

A foundation failure occurred also at the junction of this experiment and the adjoining concrete pavement. This was apparently caused by expansion of the concrete, and the brick on a strip 2 feet wide are cracked and worn. The brick of section N are rough and badly broken.

There has been little or no transverse cracking, although a considerable number of fine longitudinal cracks have developed and in some cases extend through several sections. However, they seem not to have affected the behavior of the wearing surface. Except for the repair of defects directly caused by subgrade and foundation failure, maintenance on this experiment has been confined to the filling of these cracks with bituminous material. The cost of such maintenance to date has amounted to 20.10 cents per square yard.

The brick section remains in excellent condition, showing little wear and no indication of failure under a traffic of approximately 3,000 vehicles per day.

(Continued from p. 147)

occurs also increases and the influence of vehicle speed becomes of greater importance. On a moderately rough road a vehicle of the type used will develop reactions of the general order of 150 per cent of the static wheel load with a frequency of fifteen to twenty-five times per mile when traveling at the rate of 40 miles per hour. These data further indicate that twice the static wheel load may be taken as a general maximum value of the reactions which such a balloon-tired vehicle will produce with sufficient frequency to be considered in pavement design.

CONCLUSIONS

Considering as a whole these tests with a balloon-tire-equipped bus, it may be said that -

1. Reactions of about three times the static wheel load may be encountered in isolated cases, mostly where pavement failure has already occurred and repairs are necessary.
2. A few reactions amounting to twice the static wheel load may be expected for each mile of rough surface.
3. The frequent reactions for rough surfaces amount to about 1.5 times the static wheel load.
4. Even the smoothest roads produce frequent reactions of from 1.1 to 1.2 times the static wheel load.
5. Concerning the maximum reactions produced at a large number of individual roughness conditions, the general tendency is for this maximum to increase in approximate proportion to speed up to about 40 miles per hour, and at speeds greater than that comparatively slight increases in reaction are noted. However, occasional roughness conditions may occur where the maximum reaction developed at 20 or 30 miles per hour is as great as the maximum produced by the majority of the roughnesses at only the higher speeds.

WASHING MACHINE DESIGNED FOR USE IN DETERMINING CONSTITUENTS OF FRESH CONCRETE

By William A. Blanchette, Highway Engineer, U. S. Bureau of Public Roads

IN THE analysis of fresh concrete to determine the proportion of each ingredient contained in it the ingredients must be separated. One method is to wash the concrete on a No. 100 sieve, to reclaim the cement from the wash water, to split the aggregates into fine and coarse on the proper sieve and by the difference between the initial weight of the concrete and the weight of the sum of the solids reclaimed, to calculate the weight of the water. Another method, which is based on the Archimedes principle, and which is referred to as the Dunagan method, consists of weighing the concrete in air and in water, washing out all material passing the No. 100 sieve, determining the weight of cement by differences in immersed weights, converting immersed weights of cement, fine aggregate, and coarse aggregate to air weights, and determining the weight of water by differences in air weights.

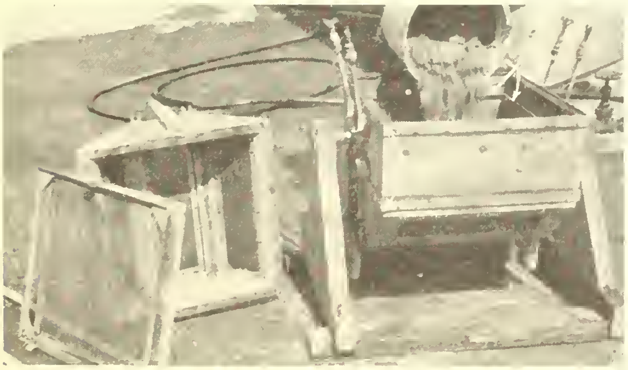


Figure 1.—The Sample of Fresh Concrete to be Analyzed is Placed on the No. 4 Sieve Which is Directly Below the Perforated Pipe and Attached to Base Frame "A." Another Sieve is Placed on Cleats "B" Above the Pipe

Either method requires that the sample of concrete be washed on sieves, and in order to facilitate this washing process a washing machine was recently devised and used successfully in connection with the Dunagan method.

As may be seen in Figures 1 to 4, this machine consists of an octagonal drum mounted on a frame. No. 100 sieves are inserted in six places around the circumference of the drum, three in the top removable cover,

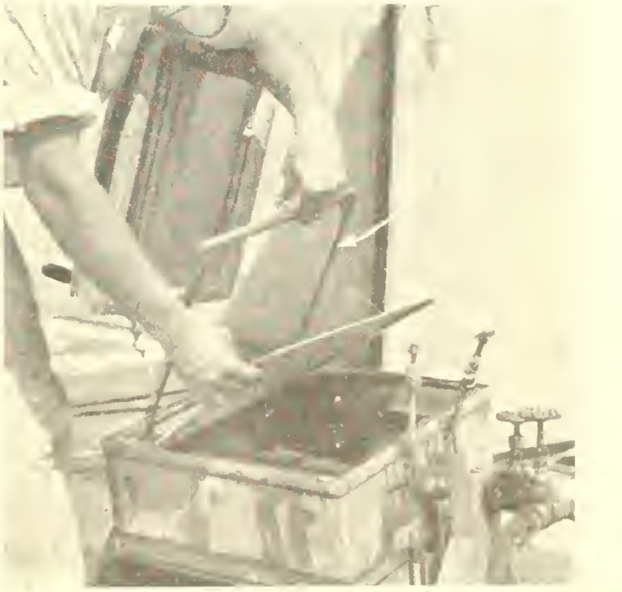


Figure 2.—Frame C is Placed over This No. 4 Sieve. This Frame Comes in Contact with the Top Cover and Holds the Sieve Firmly in Place. The Top Cover is Then Clamped in Postion and the Water Turned on

and three in the bottom cover, also removable. A perforated pipe to which the water line is coupled and through which the water is sprayed under pressure extends horizontally through the center of the drum. Two No. 4 sieves are inserted in the drum, one on either side of the water pipe. These form an inner compartment in which the sample to be washed is placed. The removable covers, containing the No. 100 sieve, are equipped with rubber gaskets and are clamped to the base frame so that all material leaving the drum must pass through the No. 100 sieve. Each piece of No. 100 sieve is reinforced on the outside with No. 4 sieve.

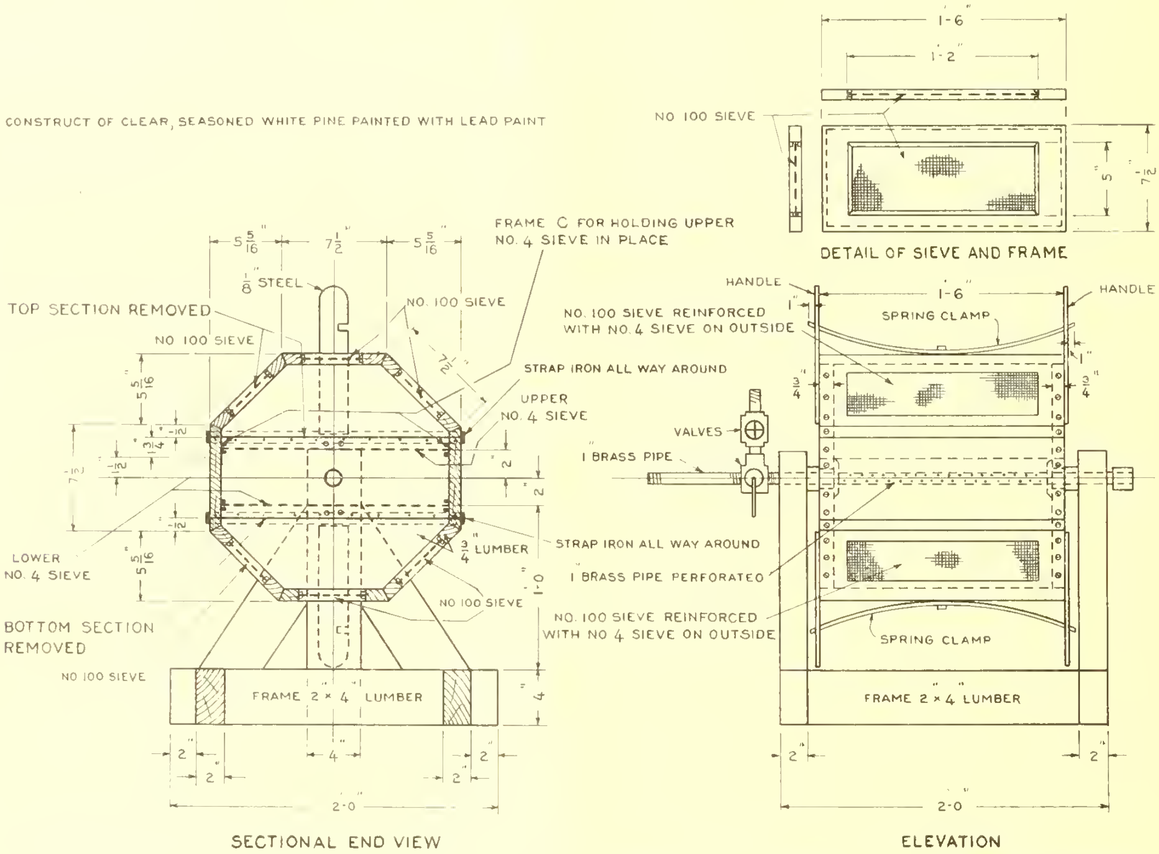


Figure 3.—Design of Washing Machine for Determining Constituents of Fresh Concrete

As may be observed in the photographs, the machine as actually constructed differs in certain details from the design shown in Figure 3. The handles attached to the covers are of wood, and the clamps with which the latter are affixed to the base frame are of a different type from those shown in the drawing.

The sample of fresh concrete is placed in the inner compartment on the lower No. 4 sieve which is attached to the base frame. The upper No. 4 sieve is then placed in position, and frame C, which holds this sieve firmly in place, inserted; thus confining the coarse aggregate so that it will not wear out the No. 100 sieve inserted in the top cover. The cover is then clamped in position, the water turned on, and the drum revolved and shaken until all material that will pass the No. 100 sieve has been washed out. By holding the drum so that the No. 4 sieves are in a horizontal plane and shaking the apparatus the fine aggregate is washed down through these sieves and is separated from the coarse aggregate. The fine aggregate is then removed in the bottom cover. After removing the upper No. 4 sieve and revolving the drum, with the top cover in place, the coarse aggregate is removed in the top cover. This machine is also used for determining the following correction factors (solids passing and retained on the 2-size sieves) which must be applied in the analysis: Fine aggregate passing the No. 100 sieve; fine aggregate retained on the No. 4 sieve; coarse aggregate passing the No. 100 sieve; coarse aggregate passing the No. 4 sieve and retained on the No. 100 sieve; cement retained on the No. 100 sieve; fine aggregate ground up during the mixing action so that it passes the No. 100 sieve.

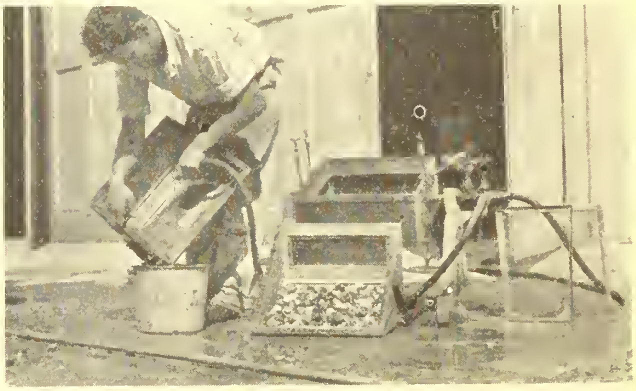


Figure 4.—After All Materials Passing the No. 100 Sieve Have Been Washed out, the Coarse and Fine Aggregates are Separated and Removed from the Machine. The Fine Aggregate is being Removed from the Bottom Cover. The Coarse Aggregate is in the Top Cover

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

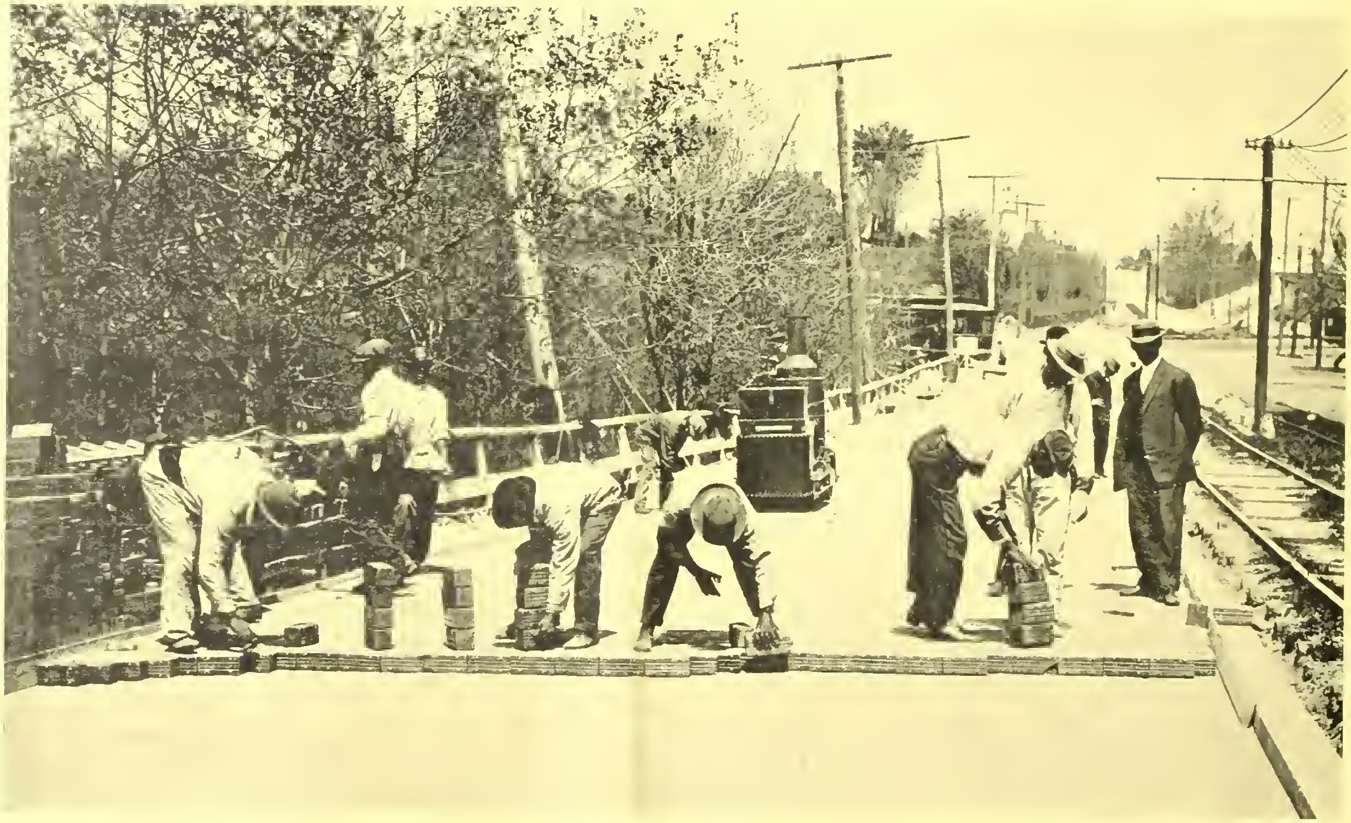
CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

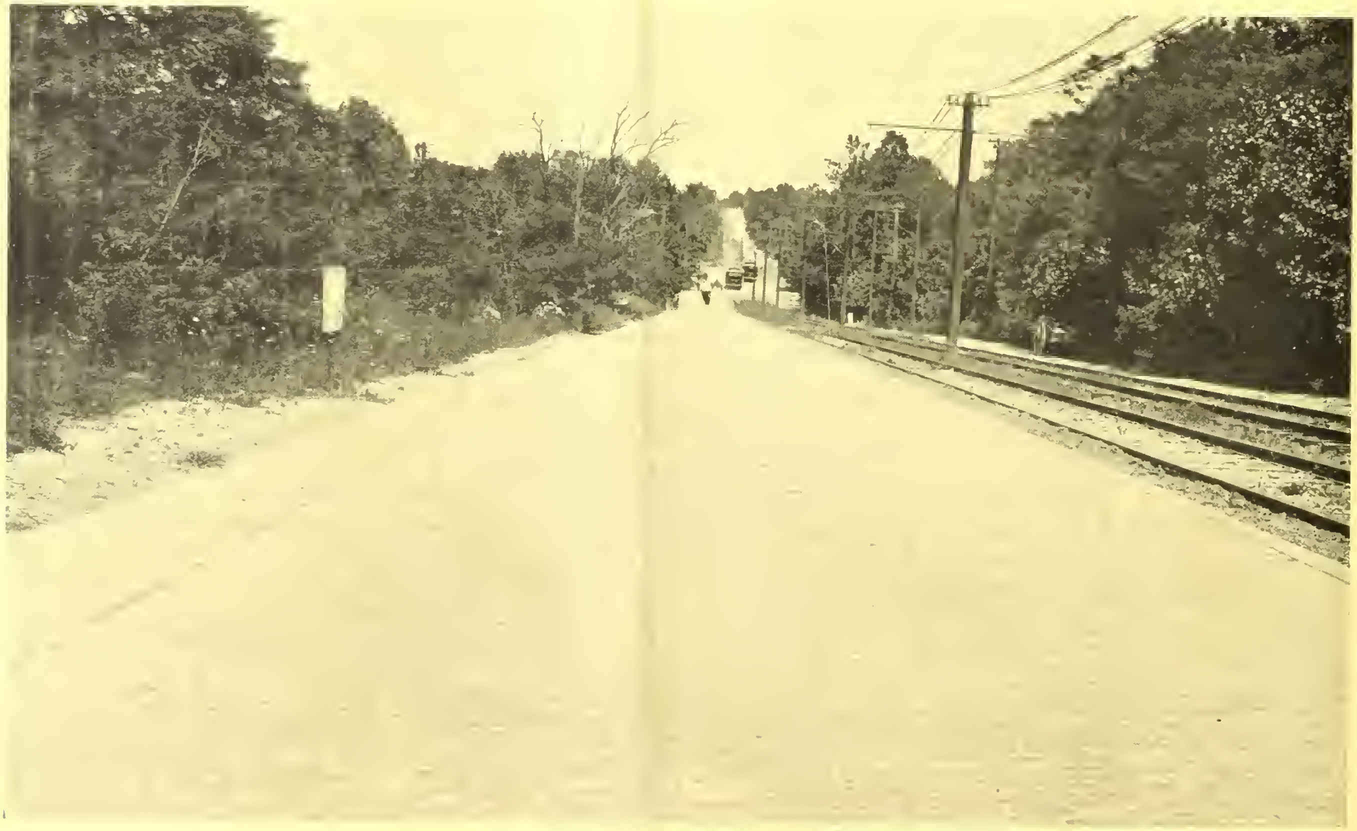
OCTOBER 31,1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,403.0	$ 2,700,450.56	$ 1,328,329.22	46	89.5	15.8	105.3	$ 975,959.82	$ 487,979.87	33.0	17.2	50.2	$ 4,261,827.01	Alabama
Arizona	1,215.5	3,251,725.93	1,699,161.37	66	114.5	117.8	232.3	209,427.52	41,885.51		32.0	32.0	247,631.71	Arizona
Arkansas	1,967.2	2,285,380.53	1,045,397.70	85	64.5	38.4	102.9	448,044.66	224,022.33	20.5		20.5	2,129,231.99	Arkansas
California	2,445.6	6,237,783.76	2,813,210.09	66	140.9	40.3	181.2	2,244,885.20	168,351.60	20.8	6.5	27.3	356,696.85	California
Colorado	1,671.8	2,987,153.64	1,542,919.32	87	110.1	62.7	172.8	133,878.96	60,245.51	9.6		9.6	1,678,307.24	Colorado
Connecticut	289.8	4,278,144.69	1,626,118.39	64	30.5		30.5	373,625.53	182,887.81	8.3		8.3	163,698.10	Connecticut
Delaware	371.5	748,723.35	334,281.00	94	18.6	2.0	20.6	269,495.80	53,200.00	10.2	9.2	19.4	88,740.82	Delaware
Florida	660.2	3,668,655.29	1,696,567.65	74	105.0		105.0	1,173,778.26	586,889.11	30.6		30.6	1,965,408.13	Florida
Georgia	3,183.6	5,745,668.03	2,502,053.46	63	118.3	147.1	265.4	1,565,830.33	553,012.17	80.0	38.5	118.5	83,106.02	Georgia
Idaho	1,298.7	3,211,402.96	1,743,857.80	90	140.6	294.2	434.8	928,590.90	218,637.19	47.3	36.0	83.3	214,700.11	Idaho
Illinois	2,874.8	23,724,195.58	10,184,987.71	83	730.0	40.6	770.6	2,937,914.13	616,883.16	97.8	4.7	102.5	513,891.73	Illinois
Indiana	1,883.2	10,328,537.06	5,131,611.19	90	387.5	15.9	403.4	48,197.89	24,098.94	4.7		4.7	343,412.82	Indiana
Iowa	3,330.4	7,782,312.52	2,461,967.61	90	371.8	81.7	453.5	977,998.29	18,423.25	38.6	1.2	39.8	9,652.78	Iowa
Kansas	3,734.6	2,852,716.39	1,288,978.89	67	165.5	143.6	309.1	2,551,455.41	543,473.56	177.8	19.2	197.0	365,781.58	Kansas
Kentucky	1,837.4	4,337,181.92	1,637,180.49	54	194.6	113.1	307.7	1,644,512.93	558,219.52	86.1	44.9	131.0	86,633.38	Kentucky
Louisiana	1,579.5	7,083,347.35	3,239,787.06	56	86.4	9.9	96.3	1,825,162.15	667,322.60	25.2	4.5	29.7	7,964.05	Louisiana
Maine	774.2	2,032,469.51	855,036.46	88	72.8		72.8	651,438.41	130,287.66	23.7		23.7	18,728.18	Maine
Maryland	805.8	1,419,044.25	672,714.42	76	56.6	4.7	61.3	528,288.35	41,220.75	17.7	1.5	19.2	20,768.90	Maryland
Massachusetts	831.9	7,301,674.93	2,573,484.99	79	79.3	5.0	84.3	550,120.53	80,000.00	10.0		10.0	15,743.13	Massachusetts
Michigan	2,224.1	7,811,835.19	3,428,567.50	75	320.5	51.2	371.7	1,069,561.73	390,977.50	67.2	17.7	84.9	790,188.17	Michigan
Minnesota	3,950.5	11,965,505.67	2,819,831.52	99	243.1	367.5	610.6	767,296.02	6,300.00	28.8	10.0	38.8	8,400.00	Minnesota
Mississippi	1,837.9	3,668,733.71	1,794,739.80	77	172.0	58.8	230.8	579,412.28	289,706.12	29.7		29.7	4,627,266.31	Mississippi
Missouri	3,106.5	3,694,249.88	1,358,220.34	76	82.3	44.6	126.9	1,548,969.71	302,219.71	93.3	2.9	96.2	310,455.24	Missouri
Montana	2,685.9	5,189,717.36	2,872,969.39	69	394.2	206.2	600.4	1,801,284.29	1,013,270.73	183.4	27.6	211.0	1,122,184.01	Montana
Nebraska	4,216.8	4,026,592.31	1,949,067.39	65	112.2	131.9	244.1	1,337,109.29	614,815.84	48.3	36.9	85.2	1,150,560.55	Nebraska
Nevada	1,334.9	1,792,520.82	1,426,209.59	80	38.4	118.9	157.3	501,415.15	66,134.78		44.0	44.0	63,225.46	Nevada
New Hampshire	424.8	752,333.19	341,340.93	69	24.8	3.4	28.2	46,112.96	18,445.18	1.2		1.2	248,048.21	New Hampshire
New Jersey	623.6	7,499,000.63	2,625,546.51	51	65.6	.5	66.1	359,376.90	124,468.87	3.8		3.8		New Jersey
New Mexico	2,245.3	1,869,960.82	941,895.83	54	92.8	56.7	149.5	698,696.99	294,099.83	48.2	42.1	90.3	590,422.98	New Mexico
New York	3,336.5	21,648,110.00	7,498,610.00	56	554.6	29.1	583.7	2,440,100.00	730,360.00	58.1	2.3	60.4	820,773.72	New York
North Carolina	2,248.1	1,478,974.04	734,458.25	57	126.0	9.0	135.0	1,836,407.57	924,917.66	258.7	17.3	276.0	2,604,253.12	North Carolina
North Dakota	5,018.0	4,523,809.23	1,788,721.71	67	464.3	528.3	992.6	1,397,931.61	753,414.66	53.6	288.4	342.0	116,523.08	North Dakota
Ohio	2,880.7	10,145,962.64	3,400,297.84	76	253.3	93.0	346.3	1,823,290.00	328,306.67	34.4	5.5	39.9	669,570.13	Ohio
Oklahoma	2,360.9	3,834,682.08	1,871,212.42	84	184.0	43.5	227.5	1,792,617.59	715,468.62	88.1	19.9	108.0	726,934.11	Oklahoma
Oregon	1,624.9	2,506,123.62	1,232,197.16	65	60.1	62.1	122.2	683,698.40	210,887.60	13.6	40.0	53.6	626,368.86	Oregon
Pennsylvania	3,089.4	9,027,731.37	3,670,388.20	66	346.5		346.5	2,693,201.15	483,604.73	77.1	5.8	82.9	819,667.01	Pennsylvania
Rhode Island	263.8	1,080,305.71	317,616.79	53	19.1	4.3	23.4	188,252.22	94,126.10	.1		.1	92,133.54	Rhode Island
South Carolina	2,001.7	2,919,748.47	1,162,992.73	52	153.1	101.1	254.2						293,595.40	South Carolina
South Dakota	4,002.6	4,752,491.41	2,194,763.65	65	384.8	299.8	684.6	128,424.30	69,435.71	26.0	.5	26.5	84,325.73	South Dakota
Tennessee	1,691.3	3,440,213.76	1,717,324.83	23	111.7	34.4	146.1	1,446,968.91	723,484.40	59.0	14.7	73.7	1,226,242.70	Tennessee
Texas	7,796.5	15,057,146.88	6,612,360.45	63	712.4	383.0	1,095.4	5,264,974.51	1,613,274.20	231.8	190.4	422.2	1,239,807.76	Texas
Utah	1,147.5	1,483,070.52	995,109.39	84	98.6	124.4	223.0	965,284.08	260,768.75	40.3	15.4	55.7	325,780.84	Utah
Vermont	358.5	849,927.15	347,113.53	89	37.7		37.7	120,039.42	10,803.56	5.4		5.4	36,721.46	Vermont
Virginia	1,918.4	3,142,504.00	1,536,142.11	64	181.5	19.3	200.8	1,643,169.56	652,111.70	73.7	15.8	89.5	823,471.32	Virginia
Washington	1,243.3	2,718,102.75	1,226,820.63	95	97.4	7.4	104.8	1,630,534.58	480,291.47	67.5	1.1	68.6	331,188.24	Washington
West Virginia	913.1	2,050,270.79	811,407.38	75	59.0	5.0	64.0	1,647,906.40	608,787.09	64.1	8.1	72.2	232,290.56	West Virginia
Wisconsin	2,589.6	9,492,129.96	3,178,900.49	86	275.2	128.0	403.2	415,831.21	82,670.62	9.6	17.8	27.4	59,329.44	Wisconsin
Wyoming	1,992.5	3,115,757.93	1,830,037.87	83	173.3	212.3	385.6	650,775.16	17,710.82	79.9	30.2	110.1	6,921.26	Wyoming
Hawaii	74.8	1,801,111.47	1,089,806.49	36	50.5	1.7	52.2	540,398.15	240,210.23	11.5	1.9	13.4	964,574.81	Hawaii
TOTALS	102,361.1	255,315,191.91	107,152,315.54	71	8,966.0	4,258.2	13,224.2	56,057,645.21	17,378,113.74	2,498.3	1,071.7	3,570.0	33,593,148.55	TOTALS
Construction completed		170,634,000.00	76,555,000.00											
Balance uncompleted		84,681,000.00	30,597,000.00											

[1] The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.



LAYING PAVING BRICK ON THE CONNECTICUT AVENUE EXPERIMENTAL ROAD, APRIL, 1913



CONDITION OF BRICK SURFACE IN JUNE, 1913

U. S. GOVERNMENT PRINTING OFFICE: 1932

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 10 DECEMBER, 1932

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 10 December, 1932

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE PROBLEM OF MOTOR VEHICLE REGULATION

WITH SPECIAL ATTENTION TO THE MOVEMENT OF HIGHWAY TRAFFIC ACROSS STATE LINES

Reported by H. H. KELLY, Division of Highway Transport, United States Bureau of Public Roads

ABSTRACT

The operation of 26,000,000 motor vehicles in the United States is subject to the diverse regulations of 48 States and the District of Columbia.

The problem of control is complicated by the large number and great variety of motor vehicles, their frequent operation outside the State of registration, and the many forms which regulation takes.

Analysis of the elements of regulation applicable to vehicles in general shows wide divergencies among the several States, particularly in regard to physical and business characteristics of vehicles, and taxation.

Even greater lack of uniformity exists in the regulation of for-hire vehicles, which presents one of the most difficult phases of the problem under discussion.

One result of this non-uniformity has been a considerable volume of court litigation, and the Supreme Court has recently handed down two decisions of basic importance in upholding new laws of Texas and Kansas.

A second result of the existing situation has been a movement toward greater standardization and rationalization of motor vehicle regulations on the part of important groups, including national and State officials, safety experts, engineers, economists, business men, and others.

The outlook for the immediate future is for further legislative action, with 43 State legislatures scheduled to meet in 1933, and with bills under consideration in both houses of the United States Congress.

Most of the elements of the problem of regulation lie within the exclusive jurisdiction of the individual States, and the goal of greater uniformity can be attained only through appropriate State action.

Regulation of interstate for-hire vehicles, and fixing of national standards for size, weight, and equipment of such vehicles, have been suggested as a proper activity of the Federal government, but measures to this end proposed thus far appear to be largely of an experimental character.

The vital need at present is for more factual data as a basis for wise legislation. Special study by each State of its own regulatory problems, and consultation with neighboring States, would seem to offer the most fruitful field for immediate State action.

Intelligent collaboration of engineers, economists, business men, safety experts, and officials of government will provide the ultimate solution to the problem of motor vehicle regulation.

A COMPLETE highway map of the United States would show a close-knit web of lines touching every part of the Nation, and representing 3,000,000 miles of road.

Moving back and forth on this vast network today are 26,000,000 motor vehicles.

Across this busy picture of modern communication lie the invisible, yet very real, boundaries of the 48 States of the Union.

Here is the essence of the American problem of motor vehicle regulation. The motor vehicle, mechanically capable of moving anywhere on the highway system of the entire nation, is subject to the diverse control of 48 different States. Although the vehicles themselves have become standardized to a large extent under mass-production methods of manufacture, and the highways over which they operate are steadily approaching a uniform standard of excellence, the laws regulating their operation are still characterized by nonuniformity.

The purpose of the present paper is to consider this whole question of motor-vehicle regulation in the United States, and to reduce its intimidating complexity to a manageable set of individual elements. Such an analysis is the first step toward comprehension of the general problem, and an essential preliminary to consideration of possible means of rationalization and standardization in the future.

GENERAL BACKGROUND

There are several reasons why the matter under discussion is a puzzling one. In the first place, it has arisen to the importance of a pressing national problem in less than a single generation, and past experience with other forms of wheeled traffic offers little or no guidance in handling it. Again, its size renders it difficult—only figures in millions or billions seem adequate to express it statistically—through the number of operating units involved, the multitude of owners and operators directly interested, the great extent of territory covered, the stupendous sums represented annually by receipts from vehicles and expenditures on highways. Finally, it contains many conflicting elements—the need for public safety and protection as opposed to the natural liberty of movement of the motor vehicle (all the more difficult to reconcile in a nation where one out of every five persons is an automobile owner, and where practically everyone is constantly or occasionally a highway user); the interest of the private-passenger automobile owner as contrasted to the interests of the commercial vehicle owner; the diverse points of view of the various kinds of commercial operators; the competitive position of other forms of transport.

Only one or two illustrations of these conflicting interests need be given.

There are more than 26,000,000 motor vehicles operating in the United States this year, but five out of every six of them are passenger cars, privately owned and operated. The balance of highway-user power now rests with the private passenger-carrying vehicle, and the needs and desires of its myriad owners have both dictated in large part the present regulatory system and have been the chief influence in the improvement of the highways of the country. The utility vehicle, the truck, has consequently taken second place in the public mind, although it is undoubtedly a more vital factor in the economic welfare of the Nation to-day than ever before. The average man, in fact, has been all too willing to condemn the truck on no more rational grounds than that it "takes up too much space on the highway," ignoring the material benefits which that same truck has conferred upon him in improved service and reduced costs of distribution.

Had motor transportation and rail transportation developed simultaneously, each would likely have filled its proper place in the economic structure with little friction, but the railroad was more than half a century ahead of the motor vehicle, and the latter has had to forge its way into an established transportation system. Thus, it has not been possible to treat the newcomer as a distinctly separate entity, desirable as that might have been from a purely economic standpoint, but, rather, it has been subject—and this is true in most modern countries, as well as in the United States—to considerations growing out of the existence of the older means of transportation. The issue is further complicated by the fact that the railroads form part of the banking and investment structure of the Nation, which has entitled them to consideration as repositories of the public wealth, rather than simply as transportation agencies subject to free competition.

Add to these conflicting elements, which are inherent in as flexible and extensive an organism as motor transport, the fact that legislative control is vested in the individual States, and the complicated situation existing to-day is seen to be only an inevitable result. Perhaps one way out of the dilemma, in the beginning, might have been the assumption of unified control over motor vehicles by the Federal Government, a policy which has simplified this matter in many foreign countries. Yet, under our system of government, with police powers reserved to the several States by the tenth amendment to the Constitution, such a plan was not considered, and to-day the problem is so large that a single nation-wide control over all vehicles is clearly impracticable.

POINT OF VIEW

The point of view of the present discussion of motor-vehicle regulation is a dual one, considering the individual State (1) as a sovereign Commonwealth and the proper source of most if not all regulatory legislation, and (2) as a member of the Federal Union of 48 States and the District of Columbia. This approach to the subject will entail a study of the situation now resulting from the efforts of the several States to regulate motor traffic, and some indication of what may be done to effect greater uniformity of regulation and greater freedom of movement of vehicles across State lines than now exists.

There is ample justification for this objective of uniformity and freedom of movement. A very large volume of traffic already exists across wide areas of the United States, regardless of State lines, and that this is worthy of encouragement is hardly disputable; this traffic, certainly as regards passenger cars, and in many portions of the country as regards trucks also, tends to balance between the individual States, so that the citizens of one State quickly receive proportionate benefits from liberality and uniformity in their own laws through increased liberality of treatment in the States which they visit. Moreover, the Nation as a whole has a vested interest in its highways everywhere, as witness the large expenditures made on Federal-aid roads under a policy of the National Government first adopted in 1916 and continued ever since. In this Federal-aid highway system are now included more than 200,000 miles of important roads. The major purpose of this system is indicated by the Federal-aid highway act of 1921 as being "the completion of an adequate and connected system of highways interstate in character." Recognition of the importance to the entire Nation of interstate commerce is contained in the Constitution itself, which provides that Congress shall have the power to "regulate commerce with foreign nations and among the several States * * *".[1]

STATISTICAL DATA

It will be helpful to glance at some figures which show both the magnitude of the problem under discussion and the complications lent to it by the numerous classes of motor vehicles.

Official reports of the United States Bureau of Public Roads give registration figures at the end of 1931 as follows: Passenger automobiles, taxicabs, and busses, 22,347,800; motor trucks and road tractors, 3,466,303; trailers, 349,930.

To segregate these totals into the individual classes of vehicles desired, the best figures available, taken from various sources,[2] have been used to compile the data given in Table 1.

TABLE 1.—*Registrations of motor vehicles in the United States at the end of 1931, classified according to use*

Type of vehicle		Number registered	Percentage of total registrations
Passenger automobiles, privately owned and operated		22,125,860	84.5
Taxicabs		140,000	.6
Busses—			
School busses	48,500		
Motor carrier busses	31,850		
Other revenue busses (electric railway, steam railroad, and sightseeing busses)	16,550		
Nonrevenue busses (hotel, industrial, and others)	2,000		
Total		98,900	.4
Trucks (including tractor trucks)		[1] 3,449,343	13.2
Trailers		349,930	1.3
Total all vehicles		26,164,033	100.0

[1] Separate figures for the three principal classes of trucks, privately owned and operated, common carrier, and contract carrier, are not available. A survey of traffic in 11 Western States indicated the proportion of common-carrier and contract-carrier trucks to all trucks to be 5.5 per cent and 8.7 per cent, respectively. These data were based on observations of traffic on rural roads rather than upon registrations. As a matter of fact all available data point to a more intense use of for-hire trucks than of private trucks and, therefore, the actual percentages of registrations of the former are certainly less than the percentages of traffic. Since the number and character of for-hire operators depend in part upon the nature of regulatory legislation in the individual State, these proportions are hardly applicable to the country as a whole, and it seems likely that the true ratio of contract-carrier to common-carrier trucks is considerably larger than the above figures indicate.

There emerge at once from the above tabulation two salient facts. Motor transportation is predominantly private in character, and differs thereby from any other form of modern communication. It is also individualistic and personal, for the more than 22,000,000 passenger cars are owned by almost as many individuals, and the National Automobile Chamber of Commerce estimates that of the 2,500,000 owners of trucks, 2,200,000 possess only one truck each.

These millions of vehicles do not remain always within the boundaries of the respective States in which they are registered. If they did, the question of regulating them would be much simpler than is actually the case. Instead, it is probable that most, if not all, of the passenger automobiles engage frequently in interstate trips, and great numbers of them in certain sections of the country must regularly cross State lines in their normal operations. A large proportion of the busses are used on interstate routes; as for the taxicabs, they do not as a rule figure in interstate operation. All this natural pressure toward interstate movement on the part of passenger-carrying vehicles, with mileage figures of large proportions, has led to fairly liberal regulations in most States, so that a private automobile to-day can travel freely in most parts of the country for at least limited periods of time without legal difficulties. The bus, being peculiarly susceptible of control by public regulatory bodies, also operates widely under the various State jurisdictions.

No one knows exactly what proportion of the trucks perform interstate service, although it is probably less than in the case of passenger cars. The fact that farmers use 26 per cent[3] of all trucks indicates that this percentage, at least, is probably used chiefly in local farm-to-market hauling. Much private hauling, as in delivery service, is also done within a limited radius.

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[1] United States Constitution, Article I, section 8.

[2] Basis of estimates as follows: (1) Passenger automobiles privately owned and operated, United States Bureau of Public Roads statistics of "passenger automobiles, taxicabs, and busses," with estimated taxicabs and busses deducted; (2) taxicabs, from Transit Journal, January, 1932; (3) busses, from Bus Facts for 1932, published by the National Association of Motor Bus Operators; (4) trucks, United States Bureau of Public Roads statistics of motor trucks and road tractors, adjusted to account for busses included with trucks. Seven States register busses with motor trucks, the number of such busses having been estimated in Bus Facts for 1932 as 16,960; this number, accordingly, has been deducted from total truck registrations and added to the total shown as "passenger automobiles, taxicabs, and busses" before segregation. Motor cycles, 101,074 of which were registered at the end of 1931, have been excluded from this table as being a minor element in the problem under discussion.

These assumptions are borne out by a statistical analysis of truck traffic made this year by the Bureau of Public Roads. This analysis was based on data obtained in exhaustive surveys of traffic in the following States, conducted by the bureau in cooperation with the State authorities: Connecticut (1922–23), Maine (1924), Pennsylvania (1924), Ohio (1925), Vermont (1926), New Hampshire (1926), and 11 Western States—Arizona, California, Colorado, Idaho, Nebraska, New Mexico, Nevada, Oregon, Utah, Washington, and Wyoming (1930–31). A study of these sources, combined with the results of an investigation of motor-truck freight transportation in 41 States and the District of Columbia, conducted in 1931 by the Bureau of Public Roads and the Department of Commerce, produced the following distribution of truck traffic:

	Per cent
Privately owned and operated	85.8
Interstate	2.6
Intrastate	83.2
Operated for hire	14.2
Interstate	4.4
Intrastate	9.8
All truck traffic	100

ELEMENTS OF PROBLEM AND EXISTING SITUATION

Having sketched in a background of basic data, and shown how the numerous kinds of motor vehicles and their movement across State lines create the first complications in our problem, we are ready to consider in detail the most important difficulty of all—that of the many forms of regulation of these vehicles.

The word "regulation" will be used in this article as a convenient term to cover all phases of vehicle control. At first glance, there may appear to be certain main groups of regulatory methods which are distinct from each other—for instance, highway protection measures, public safety measures, and business control or taxation—but upon closer inspection these are seen to be so closely interrelated that any attempt to group them in this manner only adds to the confusion. Thus, the size and weight of a vehicle may be controlled for protection of the highway pavement and bridges, but these characteristics also involve the question of public safety, and at the same time provide a basis for taxation. If, therefore, all the diverse yet interrelated forms of control be regarded simply as individual parts of a general problem of regulation, the task of analysis may be materially simplified.

The following is an attempt to set down in orderly fashion all the phases of this complex problem.

1. Forms of control applicable to motor vehicles in general:

    Registration (license plates).
    Antitheft protection (certificate of title).
    Operators' and chauffeurs' licenses.
    Public liability (financial responsibility).
    Regulation of traffic on the highway.
    Physical characteristics of vehicles
        Limitations on size and weight.
        Equipment.
    Business characteristics of vehicles
        Privately owned and operated.
        For hire.
    Taxation.
    Privileges of interstate operation (reciprocity).
    Enforcement.

2. Additional forms of control applicable to for-hire vehicles:

    Rates, practices, and service.
    Certificates of convenience and necessity for common-carrier operations.
    Hours of labor of truck and bus drivers.
    Insurance or other special financial responsibility.

To answer completely the question of what each State has enacted in the way of specific legislation on these topics would require a volume in itself. At best, there can be indicated here only some examples of the diversified statutes passed by State legislatures and now in effect.

CONTROL OF MOTOR VEHICLES IN GENERAL

Registration.—To begin with, let us visualize a new vehicle arriving from the factory—passenger car or truck or bus. It is delivered to its owner and the problem of control begins. The first thing to be done is to give it an official identity, which means registration; and to perform that job, an administrative organization is necessary. All States now have some form of registration law, and all States issue identifying license plates to vehicle owners. Beyond that, however, there is little uniformity. For example, the National Conference on Street and Highway Safety has found that the office of commissioner or registrar of motor vehicles exists at present in only 19 States; that the secretary of state performs the duties of registrar in 14 States; that the department of public works or State highway commission registers vehicles in eight States; and that in other States these duties are delegated to such widely diverse bodies as State auditors, State tax commissions, or others, and in one State even to the warden of the State penitentiary.[4] Most States require two license plates on each automobile, but some—Alabama, Florida, Mississippi, Oklahoma, for instance—require only one.[5] The period of registration is customarily for one year, but does not always coincide with the calendar year; shorter periods of registration are sometimes permitted at reduced rates. There are numerous other inconsistencies in the State laws on these and similar points, but they may be regarded as of minor importance.

Protection against theft.—Linked closely with the question of registration is that of a proper certificate of title and adequate antitheft protection. Here we encounter the first major divergence among the several States. As yet, only 22 States and the District of Columbia have some form of certificate of title law, but 31 States and the District of Columbia maintain a "stolen car" department, or special files which make possible effective action in cases of theft.

Operator's license.—Once the vehicle itself has been identified and licensed, the driver or operator who will set it in motion on the public highway enters the picture.

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[3] United States Census of Agriculture, 1930.

[4] Uniform Vehicle Code, the Basis for Nation-Wide Uniformity in State Motor Vehicle Laws, published by the National Conference on Street and Highway Safety, Washington, D. C., 1932.

[5] Much of the legislative data hereinafter cited is from a Digest of Regulations Governing the Operation of Motor Vehicles Throughout the United States and Provinces of Ontario and Quebec, Canada, 1931–32, compiled by E. Austin Baughman, Commissioner of Motor Vehicles, State of Maryland. See Appendix A.

Here appears further lack of uniformity among the States, despite the fact that the operator's license is now generally regarded as a keystone in the arch of highway safety. Only 25 States and the District of Columbia require operators to be licensed; 30, however, stipulate a special license for chauffeurs. Even more significant is the fact that in only 18 States and the District of Columbia is any examination required for applicants for operator's license; 13 of these have driving and vision tests, and written or oral queries on the motor vehicle laws. The minimum age limit for operators in 19 States and the District of Columbia is 16 years; in others the limit ranges down to as low as 12 years, and 13 States set no definite limit, the law usually permitting "any person" to be licensed.

Financial responsibility.—With the vehicle officially identified and in operation, it becomes immediately a factor in the general public welfare. Nothing can entirely guarantee it against accident, with resultant risk to life and property. The total cost of automobile accidents in the United States in 1931, including both personal injury and property damage, has been variously estimated at from $1,000,000,000 to more than $2,500,000,000. Inevitably there has arisen a demand for some sort of public liability protection on the part of owners and operators of motor vehicles which will tend to insure payment of compensation for injury or damage.

One State, Massachusetts, has tried the plan of compulsory insurance for all drivers, but its success has been questioned, as the number of motor vehicle accidents reported has increased rather than diminished in that State since passage of the law, and insurance rates have risen. On the other hand, 19 States have adopted legislation along the lines of the so-called financial responsibility law which provides: (1) That any person convicted of a major traffic offense must, before his permit is restored, show financial responsibility for the sum of $11,000, of which $1,000 is for property damage and $10,000 for injury to one or more persons in an accident, by posting collateral, taking out bond, or obtaining liability insurance; (2) that a motorist failing to pay a judgment levied against him by a court of competent jurisdiction up to this amount for an accident shall be deprived of his permit to drive until he meets the requirement of the law; (3) that all States recognizing this law maintain reciprocal relationship, prohibiting a motorist convicted in one State from operating in another. The following States have enacted safety responsibility laws: California, Connecticut, Delaware, Indiana, Iowa, Maine, Maryland, Minnesota, Nebraska, New Hampshire, New Jersey, New York, North Carolina, North Dakota, Rhode Island, South Dakota, Vermont, Virginia, and Wisconsin.[6]

In addition to compulsory insurance and financial responsibility as methods of insuring protection for the public, other plans have been advanced from time to time, but have not been embodied in law. An example is the so-called "Marx plan" under which a State compensation fund would be created by compelling all vehicles to contribute a premium to such fund in order to compensate victims of accidents in a manner similar to the workmen's compensation act. Another plan would require compulsory insurance, but with rates depending upon the accident record of the insured and with damage claims handled through State-appointed commissions rather than the civil courts. These various plans, being untested by actual experience, remain controversial.

Traffic regulation.—The basically important question of the regulation of traffic, with its attendant potential benefits in reducing the tragic toll of accidents on streets and highways, has received attention to some degree in every State. More or less complete regulations exist on such matters as rule of the road, speed restrictions, pedestrians' rights, traffic signs and markers, etc., although there are considerable divergencies among the several States. For instance, 31 States and the District of Columbia have fixed definite speed limits for the operation of private passenger cars in the open country, these varying from 30 to 50 miles per hour, and have usually set lower limits for commercial vehicles. Fourteen States require that vehicles be driven at a "reasonable and proper speed," to quote a phrase frequently employed, although certain of them fix a limit beyond which higher speed is deemed "prima facie evidence of improper driving." Three States set no limitations to speed. In addition to State laws governing speed, most municipalities have their own regulations.

Eight States have declared it unlawful to throw lighted cigarettes out of a motor vehicle. Passing a street car on the left is strictly prohibited in 32 States. Forty States require drivers to use hand signals to indicate their intention of turning or stopping, but the nature of these signals is not uniform.[7]

Up to this point in our consideration of the various phases of motor vehicle control, we have been dealing with those matters which place basic administrative safeguards upon vehicles in the interest both of owners and of the public welfare. Now we enter a field where uniformity is even more lacking, and yet where regulation may completely put a stop to certain forms of motor transport. In this field are the questions of control over the physical and business characteristics of vehicles, their taxation, and the privilege, or lack of privilege, of operation outside the State of registration.

Physical characteristics.—It is necessary that limits should be fixed to the size of the larger vehicles to prevent their reaching dimensions which would actively endanger every other user of the highway. Limits have, in fact, been fixed by most States, but in many cases they reveal small trace of being based upon scientific or factual data. In the laws now in effect[8] practically the only dimension on which there is general uniformity is that of width, 38 States and the District of Columbia having set this figure at a maximum of 96 inches; but Florida limits it to 84 inches (except in the case of for-hire vehicles); North Carolina, Oklahoma, and South Carolina to 90 inches; and Maryland to 93 inches. A few States permit widths in excess of 96 inches. The maximum permissible height varies from 11 feet to 14 feet 6 inches, with 15 States and the District of Columbia uniting on 12 feet 6 inches, and with no regulations in effect in nine States. The maximum permissible length for a single vehicle varies from 26½ to 40 feet. The maximum permissible length for a combination of vehicles varies from 30 to 85 feet, with 13 States and the District of Columbia uniting on the latter high figure. The permissible number of vehicles in a train or combination of vehicles varies from two to four, with no regulations in 13 States and the District of Columbia.

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[6] Report of the Committee on the District of Columbia, U. S. House of Representatives, to accompany Senate bill 3053, 72d Cong., May 9, 1932, p. 5. Virginia this year entered the list of States having financial responsibility legislation.

[7] Summary of Motor Vehicle Regulations, 1932 edition, issued by the American Automobile Association and the Albert Russel Erskine Bureau for Street Traffic Research, Harvard University.

[8] See Appendix B.

The greatest variance of all exists among the various States in the matter of weight. Factors of wheel load, axle load, or load per inch of tire width enter into the calculations, but gross vehicle weight is usually the final consideration, and this varies in the case of a pneumatic-tired, 4-wheel vehicle from 15,000 to 36,000 pounds. In the case of a pneumatic-tired, 6-wheel vehicle, the range is from 16,000 to 48,000 pounds. A typical combination of vehicles, including a 4-wheel tractor, 4-wheel semitrailer and 6-wheel full trailer, has maximum weight limits (on pneumatic tires) varying from 24,000 to 102,000 pounds, but at least 13 States do not permit operation of a combination of units of this size.

The various items of equipment required in the several States indicate similar lack of uniformity. As a single example, a recent compilation [9] of State regulations on clearance lights shows great variety. California requires a blue clearance light in front, red or green in the rear; Delaware exacts a green light in the rear, white light in front; Illinois a yellow or amber reflector in front, a red reflector in the rear; Iowa, white marker lights or side reflectors; and so on. In actual practice this has sometimes meant changing the lighting equipment of large interstate vehicles at each State line crossed. The whole question of the equipment of the larger types of vehicle, particularly those used in common-carrier operations, upon which the various State regulatory bodies have issued many diverse rulings, is especially confused. Braking requirements are somewhat less involved, with the usual formula calling for brakes "adequate to stop within a safe distance and hold the vehicle." In some States, however, particularly those in which periodic inspection of vehicles is held, the safety of brakes is determined by their ability to stop the vehicle within a specified distance at a given rate of speed. Safety-glass equipment is a matter of comparatively recent origin, and few States have taken action upon it as yet.

Control over the physical characteristics of vehicles is also being extended in some States to cover a compulsory periodic inspection, to which all owners must submit their cars at stated intervals, usually once a year. Ten States now require such inspection, although one of them limits it to busses and another to city operations.

Business characteristics.—Control of vehicles according to their business characteristics comprises one of the most difficult elements of the problem. Each State has considered the question primarily from the angle of its own needs and conditions, with political and competitive interests sometimes the controlling factors. The business characteristics of a vehicle depend, of course, upon the nature and purpose of its use, and a primary division of vehicles would separate them into two classes—those privately owned and operated and those operated for hire. This division, entirely arbitrary from an engineering standpoint, for the roads and bridges feel no difference between a private and a for-hire vehicle, has been accepted as a proper distinction in most States of this country and in many foreign countries, under the assumption that an operator using the public highways as a place of business for direct profit to himself should pay more for these facilities than the private operator who carries his own goods or merchandise, or his own family or friends, in vehicles owned and operated by him only incidentally to his major business. Under the for-hire class have grown up two main divisions, common-carrier and contract-carrier. The common carrier of persons (the bus) is apparently the simplest type of these vehicles to control; the carrier of goods or merchandise (the truck) is more elusive, and raises questions of control which are not easy to answer.

Consideration of these business characteristics of motor vehicles brings uppermost the question of the competition of for-hire vehicles with other forms of transport, notably the railroads, which has influenced much of the legislation now found in many States. There has been a tendency to regard rail transport and motor transport as more or less identical in character, and therefore properly subject to similar forms of control. This assumption, however, ignores the basic difference between them, which is the difference between essentially monopolistic and essentially individualistic enterprises. The railroad represents a permanent investment of large capital; its operations are confined to a fixed right of way, of which it enjoys exclusive ownership and use; once established, it partakes of the nature of a natural monopoly, and its service and rates are matters of primary concern to the communities it serves. The motor carrier, on the other hand, is in its essence an individualistic enterprise, open to operators with small capital, not limited to a fixed right of way, possessing great flexibility in the matter of operating units and routes of haul, performing chiefly short-haul service, depending for its existence upon the immediate value of the services it renders, and subject always to keen competition (which is a natural and effective check upon its rates and services) from other kinds of vehicle operators as well as from rail and other forms of transport. In fact, the only form of motor transport which can be logically compared with railroad transportation, from the standpoint of public control, is that which deliberately assumes some of the attributes of the latter—as, for instance, the common-carrier bus, which must operate over a fixed route and give regular service in order to attract profitable patronage. These observations illustrate some of the difficulties which are being encountered in arriving at satisfactory solutions to the new problems created by for-hire motor vehicles.

Parenthetically, but as a means of showing truck traffic in its true relation to the business of transportation as a whole, it may be said that the Interstate Commerce Commission has recently estimated that in the peak year 1929, when all forms of traffic were at record levels, only 4.2 per cent of the inland traffic of the United States was carried by intercity trucks.[10] No estimate was made of the proportion of this small percentage handled by for-hire operators as distinguished from private operators. United States Army transportation experts have estimated that not over 5 per cent of the service of the railroads could be replaced by highway motor transportation at the present time.[11]

As previously indicated, highway transport is predominantly private in character, and its competition with other forms of transport must be ascribed in the main to privately owned and operated passenger cars and trucks.

The widely diversified nature of State legislation regarding the business characteristics of motor vehicles makes analysis difficult. Privately owned and operated vehicles generally enjoy liberality of treatment, in so far as the business nature of their operation is concerned, although we shall later see in discussing recent legislation in Kansas that business control of certain classes of private vehicles has also begun. With regard to for-hire vehicles, however, most States have adopted legislation which definitely affects the common-carrier type of operation, and in numerous cases the contract carrier as well. These matters will be further discussed in considering these special classes of vehicles (see p. 159).

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[9] Printed in Commercial Car Journal for March, 1932.

[10] Interstate Commerce Commission Report No. 23400, p. 403.

[11] Address by Ezra Brainerd, jr., member of the Interstate Commerce Commission, before the National Association of Railroad and Utilities Commissioners, Richmond, Va., Oct. 20, 1931.

Taxation.—Motor-vehicle taxation in the United States to-day presents a picture of almost complete nonuniformity.[12] The principal taxing jurisdictions have been the States and municipalities; but this year the Federal Government has also entered the field, for the first time since abolition of the war-time excise taxes, with a tax on gasoline and oil and new excise taxes on motor vehicles, parts and accessories, and tires. The two principal types of taxes levied are "general" and "special." Under the former classification fall the ad valorem taxes imposed by many States on the motor vehicle as personal property; under the latter are the annual registration and license fees, motor-fuel taxes, and similar impositions. The annual registration and license fees have diverse bases: Weight, horsepower, cost price or value, piston displacement, tonnage capacity, sometimes an arbitrary flat rate, and sometimes a combination of certain of the above factors. Such vehicles as trailers, hearses, and ambulances often have special rates. For-hire vehicles, frequently divided into numerous classes according to type of operation, are assessed extra taxes in many States.

As a particularly striking example of nonuniformity in taxes, it has been calculated[13] that the range of annual registration fees on a privately owned and operated 3-ton truck (net weight 7,000 pounds, capacity 6,000 pounds, gross weight 13,000 pounds, 30 horsepower, value $4,500, equipped with pneumatic tires) is from $9.60 in Maryland to $134 in New Mexico, with $66.36 as the average for the country as a whole; and upon the same type truck when operated as a common carrier, the range is from $18 in Missouri to $900 in North Carolina, with $271.95 as the nation-wide average. The two average figures cited are for the year 1931; and when compared with the figures for 1923, respectively $53.50 and $90, they demonstrate the rapid rise in the taxation of trucks in general and of common-carrier vehicles in particular. These fees are in addition to the usual gasoline taxes, the estimated nation-wide average of which in 1931 on this typical 3-ton truck was $95.55 for the private vehicle and $193.23 for the common carrier, the difference being due to higher mileage of the latter. The taxes cited do not include personal property, municipal, franchise, wheelage, or other local taxes.

The volume of taxes paid by motor vehicles in the United States is indicated by the estimated figure of $1,010,000,000 for 1930, this impressive total being made up as follows: Registration fees, $350,000,000; gasoline taxes, $515,000,000; municipal taxes, $20,000,000; personal property tax, $125,000,000.[14] For 1931, the grand total was estimated at $1,025,735,000 by the National Automobile Chamber of Commerce, the increase being due entirely to higher receipts from gasoline taxes, which offset decreases in some of the other categories.

Privilege of operating outside State of registration.—The extent to which motor vehicles registered in one State may operate in another naturally depends upon the character of legislation in the individual States in question. Since this legislation usually possesses little uniformity, the motor vehicle may be confronted by a large number of conflicting regulations as soon as it leaves the State in which it is registered. If all laws in each State were enforced to the letter against all vehicles entering from other States, travel by highway across State lines would unquestionably be much reduced. To avoid some of these complications a system of reciprocal arrangements has grown up among the States, based usually on the principle of "doing unto others as others do unto us." Application of this principle, however, is contingent upon the laws of the individual States; thus, one Commonwealth may have rigorous limitations as to length of time reciprocity is granted to private passenger cars, or as to the size and weight of vehicles, while another has more liberal regulations on these matters; the movement of vehicles between two such States is, therefore, automatically reduced to the terms of the more restrictive legislation.

Recent digests of State laws[15] show many interesting facts regarding existing reciprocal arrangements. The most liberal provisions apply to private passenger automobiles, and about half of the States offer unconditional reciprocity (which means freedom of operation throughout the current registration year of the State of origin) to all passenger automobiles from other States granting similar privileges. In other Commonwealths the period of free operation is limited to 30, 60, or 90 days, or in one case (New Mexico) to 6 months. Certain States require foreign visitors to obtain nonresident stickers for their cars.

In the case of trucks and other commercial vehicles the regulations are much less liberal. Among the States offering no reciprocity whatever to private or for-hire commercial operators are: Arizona, California, Colorado, Florida (except upon special permit), Idaho, Oklahoma, and Tennessee (except for occasional trip haul). Other States place heavy restrictions upon the commercial vehicles entering from "foreign" jurisdictions; for example, Arkansas limits the number of trips by private operators to four per month, and allows no reciprocity on for-hire vehicles. Idaho permits commercial cars to operate for profit, not to exceed three months, under temporary license at one-eighth the annual fee per month. Kansas requires for-hire vehicles to register if operated more than 10 consecutive days. Maine offers reciprocity only on vehicles up to 1½ tons' capacity. Maryland stipulates that foreign commercial vehicles can not carry on intrastate business or operate over fixed routes or between fixed termini. Michigan allows private trucks 10 days' reciprocity, but requires that all public carriers operating over fixed routes or between fixed termini must register.

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[12] Included in the source material on this subject may be cited A Study on Motor Vehicle Taxes, published by the American Automobile Association, Washington, D. C., in 1931, and Special Taxation for Motor Vehicles, 1931 edition, of the Motor Vehicle Conference Committee, 366 Madison Avenue, New York City.

[13] Facts and Figures of the Automobile Industry, 1931 Edition, p. 40.

[14] Special Taxation for Motor Vehicles, 1931 Edition, Motor Vehicle Conference Committee.

[15] Baughman compilation (see footnote 5, p. 155) and Summary of Motor Vehicle Acts, 1932-33, Automobile Club of Southern California.

Enforcement. The final element in the problem of regulating motor vehicles in general is that of enforcement of the motor vehicle laws. This duty has always rested exclusively in the hands of the individual States. State highway patrols or inspection forces now exist in 30 States, with as many as 475 officers employed in a single State. Eighteen States apparently have no special police force charged with enforcing their motor-vehicle regulations, these duties usually being entrusted to the regular peace officers.

SPECIAL CONTROL OF FOR-HIRE VEHICLES

Although the problem of regulating motor vehicles in general is difficult enough in itself, there is a special class of vehicle which has created a set of additional perplexities all its own. These are the for-hire vehicles, including common carriers and contract carriers of persons or property or both.

These for-hire vehicles offer a distinct service to the public, and unquestionably have become a permanent factor in American economic life. The busses, depending for their successful operation upon regularity of service over fixed routes, have lent themselves readily to control by State regulatory bodies, and all States except Delaware now regulate these common carriers of persons. The bus transportation industry is well organized, and its efforts to promote both the public safety and the interests of its members have been noteworthy.

For-hire trucks, on the other hand, present a different picture. Although many of them are common carriers, an even greater proportion are contract carriers and do not hold themselves out indiscriminately to serve the general public. The line of demarcation between these two groups is not always clear; and, because of the inherent flexibility of the motor vehicle and its mode of operation, pressure in the form of severe regulation may promptly transform the common-carrier operator into a contract carrier, or the contract carrier into a private owner. The industry is not well organized, and the divergent interests of the for-hire operator and the private owner, the common carrier and the contract carrier, the over-the-road hauler and the local truckman have further confused the situation. At the same time, the weight of conflicting State legislation, of heavy taxation, and of lack of reciprocal privileges bears most heavily upon these carriers who are in the business of highway transportation.

The Interstate Commerce Commission has summarized the present State regulation of common and contract carriers by motor vehicles as follows:

> A survey of the statutes of the 48 States (excepting Delaware, which has no regulatory body nor regulatory act dealing with carriers) reveals that 47 States and the District of Columbia have laws regulating the operation of motor vehicles as common carriers of passengers, 39 States and the District of Columbia have laws regulating the operation of common carriers of property, and 34 States have enacted laws dealing with the operation of contract carriers. Concerning this last group, 23 States have laws dealing with the operation of contract carriers which are in effect, 4 States have made attempts which have been declared invalid by the courts chiefly because of an attempt to subject the contract carrier to the same type of regulation applicable to the common carrier, and 7 States have laws which, though not yet attacked in the courts, seem to fall into the same errors as the other voided legislation.[16]

Even the bodies in which administrative control over public carriers is vested vary with the different States. A compilation made this year[17] shows, for example, that common-carrier motor vehicles are under the control of the railroad commissions in 14 States, while in others the regulatory bodies include public service or public utilities commissions, corporation commissions, departments of public works, etc.

In those States which control the operations of common carriers, a certificate of convenience and necessity is usually required of such operators before they can begin intrastate operations. Application for a certificate is made by the carrier to the properly constituted authorities, listing the routes to be covered, schedule of operations and rates or fares, vehicles to be used, and similar data, and a hearing is held at which the public and competitive interests are invited to appear. Decision is then made as to whether or not the desired certificate is to be granted. Since the operator has identified himself as desiring to perform common-carrier service, his legal status is fixed, and the process of control over him offers no insurmountable difficulties.

With the contract carrier the case is different. His service is not to the public but to individual employers, and private contracts, rather than public and competitive interests, govern his operations. Some contract-carrier service is of a regular character, an example of this being the contract trucking for chain groceries in some localities; some of it is irregular in nature, as, for example, the moving van, and similar activities which are of nonrecurring character. In addition, there is an undoubtedly large number of private owners who may occasionally haul for hire: The farmer who carries his produce or livestock to market and takes back any available pay load, or the drayman who may intermittently perform over-the-road service. Control of such operations, or even their detection, sets up manifestly grave administrative difficulties.

The above considerations apply to intrastate operations. When interstate service is performed—such service having been defined as "commerce between any place in a State and any place outside thereof: or between points within the same State but through any place outside thereof"[18]—the constitutional guarantee regarding the freedom of interstate commerce takes effect, and applications even of common carriers for certificates of convenience and necessity in interstate operations have been pronounced outside the power of State authorities to deny. State laws frequently make specific exemption to this effect. Nevertheless, the Public Utilities Commission of Ohio recently denied an application for an interstate certificate on the grounds that the highway over which service was to be performed was already congested by traffic. Its action was upheld by the Ohio Supreme Court, but appeal to the United States Supreme Court was taken by the applicant.

Hours of labor of bus and common-carrier truck drivers have been limited in a number of States by rulings of regulatory bodies, and in some cases by statute. Arizona this year enacted a statute making it a misdemeanor for any operator of a for-hire vehicle to remain on duty longer than 12 hours in any 24-hour period, except in case of emergency "where life or property is in imminent danger," and then only up to a maximum of 15 hours.

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[16] Interstate Commerce Commission Report No. 23400, 1932, Appendix F, p. 110

[17] Bus Facts for 1932, p. 47.

[18] House of Representatives bill 12739, "to regulate the transportation of passengers and property in interstate and foreign commerce by motor carrier operating on the public highways, and for other purposes," introduced June 21, 1932, by Representative Rayburn, chairman of the House Committee on Interstate and Foreign Commerce.

Every State which regulates motor carriers, directly or indirectly, requires insurance or other financial responsibility as a measure of public protection.[19]

On many of these varied questions the States are constantly adding to the volume of legislation already accumulated, or are making substantial revisions. Thus, during the first half of 1932 changes have been made in the regulatory laws of Kentucky, Mississippi, New York, Rhode Island, and Virginia, and these changes were mainly in the direction of tighter restrictions on sizes and weights and on operation of commercial vehicles. Perhaps the most drastic of all is the new Kentucky law [20] which entirely prohibits the operation of full trailers, limits the length of a single vehicle to 26½ feet and that of a tractor-semitrailer combination to 30 feet, reduces the maximum permissible weight of either a single vehicle or a tractor-semitrailer combination to 18,000 pounds gross, places both contract carriers and common carriers under the jurisdiction of the State tax commission, and imposes mileage fees upon them which are applicable also to interstate operators.

New problems also are continually arising, upon which various States take action as the need to do so becomes apparent. The recent rapid growth of the "share expense" enterprises in tourist transportation, by which small parties of persons agree to defray the expense of travel in private automobiles, is a case in point. At the annual convention of the National Association of Motor Bus Operators in Chicago this year it was reported that the States of California, New Mexico, and Louisiana now regulate this form of travel.[21]

THE JUDICIARY ANGLE

The first result of this intricate and conflicting mass of legislation in the several States has been a large volume of litigation, in which interests affected by the new laws have often sought injunctions against their enforcement. The courts have been continually called upon to interpret the new laws, and cases have frequently been carried to the Supreme Court of the United States for final decision.

Two of the most important recent opinions were handed down by the Supreme Court simultaneously on May 23, 1932, covering certain aspects of new Texas and Kansas laws.[22] Both of these State laws were upheld by a unanimous decision of the Supreme Court, Mr. Chief Justice Hughes delivering the opinion in both cases.

One of the unusual provisions of the Texas law in question was that fixing a pay load of 7,000 pounds as the maximum permitted on any "commercial motor vehicle" (which the act defines as one designed or used for the transportation of property), truck tractor, or trailer operating outside of an incorporated city or town; with the additional provision, however, that this maximum permissible load may be increased to 14,000 pounds in the case of vehicles "used only to transport property from point of origin to the nearest practicable common carrier receiving or loading point or from a common carrier unloading point by way of the shortest practicable route to destination, provided said vehicle does not pass a delivery or receiving point of a common carrier equipped to transport such load," or when used to transport property "from point of origin to point of destination" when the latter is less distant from the point of origin "than the nearest practicable common carrier receiving or loading point equipped to transport such load." On this point the opinion states:

> It is said that the exception was designed to favor transportation by railroad as against transportation by motor trucks. If this was the motive of the legislature, it does not follow that the classification as made in this case would be invalid. The State has a vital interest in the appropriate utilization of the railroads which serve its people as well as in the proper maintenance of its highways as safe and convenient facilities. The State provides its highways and pays for their upkeep. Its people make railroad transportation possible by the payment of transportation charges. It can not be said that the State is powerless to protect its highways from being subjected to excessive burdens when other means of transportation are available. The use of highways for truck transportation has its manifest convenience, but we perceive no constitutional ground for denying to the State the right to foster a fair distribution of traffic to the end that all necessary facilities should be maintained and that the public should not be inconvenienced by inordinate uses of its highways for purposes of gain. This is not a case of a denial of the use of the highways to one class of citizens as opposed to another or of limitations having no appropriate relation to highway protection. It is not a case of an arbitrary discrimination between the products carried, as in the case of Smith *v.* Cahoon, 283 U. S. 553, 567. The provision of section 7 permitting increased loads under the stated conditions applies to all persons and to all products. The discrimination is simply in favor of short hauls and of operations which, as the District Court found, are confined to small areas and greatly reduce the danger of traffic congestion and highway casualties.

On the general question of State powers to fix motor-vehicle regulations, the same opinion states:

> In exercising its authority over its highways the State is not limited to the raising of revenue for maintenance and reconstruction, or to regulations as to the manner in which vehicles shall be operated, but the State may also prevent the wear and hazards due to excessive size of vehicles and weight of load. Limitations of size and weight are manifestly subjects within the broad range of legislative discretion. * * * When the subject lies within the police power of the State, debatable questions as to reasonableness are not for the courts but for the legislature, which is entitled to form its own judgment, and its action within its range of discretion can not be set aside because compliance is burdensome.

In the Kansas case the State law was also upheld. Here the Supreme Court findings are of especial significance because the Kansas law represents one of the first attempts by a State to regulate the business operation of private motor vehicles on its highways. The act in question contains interesting definitions. A public motor carrier of property or passengers is defined as one transporting for hire as a common carrier, having fixed termini or route; a contract motor carrier of property is defined as one who is not a public motor carrier and is engaged in transportation of property for hire as a business; and a private motor carrier of property is defined as one transporting "property sold or to be sold by him in furtherance of any private commercial enterprise." Public motor carriers in intrastate commerce must obtain certificates of convenience and necessity, and contract motor carriers and private motor carriers of property, either in intrastate commerce or in interstate commerce, must obtain licenses from the public service commission. In addition to license fees, all three classes of carriers must pay a tax of five-tenths mill per gross ton-mile, keeping records to certify to the ton-miles traveled monthly. No certificate or license is to be issued by the commission until a liability insurance policy approved by the commission has been filed. The act does not apply to (1) motor carriers operating wholly within any city or village of the State, (2) private motor carriers operating within a radius of 25 miles beyond the corporate limits of such city or village, (3) the transportation of livestock and farm products to market "by the owner thereof or supplies for his own use in his own motor vehicle," and (4) the transportation of children to and from school.

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[19] For further discussion of these intricate phases of for-hire carrier regulation the interested reader is referred to the Interstate Commerce Commission Report on Docket 23400, already cited; an article on State Regulation of Motor Vehicles Operating in Interstate Commerce, by Mac Asbill, in American Bar Association Journal, February, 1931; and an article on Regulation of the Contract Motor Carrier Under the Constitution, by La Rue Brown and Stuart N. Scott, Harvard Law Review, February, 1931.

[20] Kentucky Acts, 1932, chs. 104 and 106.

[21] New York Times, Oct. 9, 1932.

[22] Opinions of the Supreme Court of the United States, No. 826, October Term, 1931 (Texas) and No. 677, October Term, 1931 (Kansas).

As regards the tax imposed on "private motor carriers of property," the Supreme Court opinion holds that "requirements of this sort are clearly within the authority of the State which may demand compensation for the special facilities it has provided and regulate the use of its highways to promote the public safety. Reasonable regulations to that end are valid as to intrastate traffic and, where there is no discrimination against the interstate commerce which may be affected, do not impose an unconstitutional burden upon that commerce." As for the distinction between operations under or over a radius of 25 miles of the private carrier's home city or base, "no impropriety in assessment or in collection * * * or denial of remedy, is disclosed."

These decisions by the highest tribunal of the Nation are of fundamental importance and may vitally influence future legislation, since they uphold the powers of State legislatures to regulate minutely the operations of commercial motor vehicles and justify the taking into consideration of competitive interests, such as railroads.

Numerous decisions have also been made by lesser courts on regulatory questions, but space does not permit of any discussion of them here. Rulings of regulatory bodies and opinions of State attorneys general have further affected the business phases of vehicle regulation in many States. The effect in general has not been one of clarification or simplification of the basic problems of motor vehicle regulation.

EFFORTS TOWARD UNIFORMITY

The second result of the existing state of regulatory legislation has been a countermovement toward uniformity and greater liberality in some noteworthy quarters.

The most serious work performed to date on certain fundamental aspects of the problem has been done by the National Conference on Street and Highway Safety, which held sessions in 1924, 1926, and 1930, at which a large number of influential delegates assisted in drawing up a uniform vehicle code for consideration and enactment by all States. To the same general end, that of promoting safety on streets and highways, the American Automobile Association, and the National Safety Council are devoting a large part of their attention.

Among State officials who administer the vehicle laws and are, therefore, at handgrips with the problem, a movement toward greater uniformity is under way in certain parts of the country. In the Central States especially much thought has been given to the question in a series of conferences at which as many as 13 States have been represented. In 1931, at a meeting in Detroit, a committee on uniformity of the motor vehicle commissioners of the Central States was organized and has since held sessions from which have grown a set of recommendations to be presented to State legislatures. The Eastern Conference of Motor Vehicle Commissioners, in which 22 States are officially represented, has arranged to invite the motor vehicle commissioners of all other States in the Union to join with them in the discussion of questions of uniform legislation and reciprocal relations.

Moreover, State administrative officials, facing a dilemma between their duty to enforce strictly the provisions of rigorous laws and the inevitable repercussion such enforcement brings upon their own commercial vehicle operators entering neighboring States, have sometimes found it necessary to enter into emergency reciprocity agreements with their neighbors as the only means of preventing a complete breakdown of truck traffic. The trouble has usually begun when one State has started to impound all trucks from other States and to hold them until a license fee is paid for each vehicle or until the owners or drivers pay fines. In retaliation, the other States affected have followed suit, and a sort of border warfare has resulted until stopped by mutual agreement. Each experience of this kind has illustrated anew the vital part played in economic life by the commercial vehicle and the natural pressure of motor-vehicle transport across State lines. The sum total of such experiences is embodied in the tolerant attitude of many States toward at least the foreign privately owned and operated vehicle—an attitude which, it may be safely said, would become more pronounced were it not for the constant changes being made in State regulations and the consequent impossibility of maintaining a permanent administrative policy.

The National Association of Railroad and Utilities Commissioners has been considering motor transportation problems for a number of years, having deemed the question sufficiently important to appoint a special committee on motor-vehicle legislation as early as 1925, in addition to a regular committee on motor-vehicle transportation. The annual conventions of the association have given prominence to discussion of these subjects.

Another group of officials considering these matters is the American Association of State Highway Officials, the membership of which includes the highway and bridge engineers of the several States. This association, through technical committees, is working toward the establishment of standard principles in highway construction, administration, finance, and use.

More than 40 agricultural, industrial, and trade organizations, comprising a large percentage of shippers using all forms of transportation, together with automobile clubs and various branches of the automotive industry, have recently joined in the formation of a National Highway Users' Conference with the purpose of consolidating and strengthening the efforts of all groups interested in motor transport. The same interests are assisting in coordinating the efforts of local organizations of similar character to deal with the legislative situation in each State. Another influential body working toward uniformity in regulations is the Motor Vehicle Conference Committee.

Within recent weeks national committees have been formed to study broad questions of transportation, and in particular the relationship between highway and rail transport.

Automotive engineers are beginning to contribute much clarification to the mechanical problems involved, and the Society of Automotive Engineers is now preparing recommendations on some of the points relating to size and weight of vehicles. Engineers of the United States Bureau of Public Roads are studying certain technical phases of the problem, including impact of vehicles on highways and the consequent effect upon the road surface about which so many unscientific and irrational opinions have been advanced in the past. Scientists at the United States Bureau of Standards are working on questions of adequate lighting and braking for vehicles.[23]

Commercial vehicle operators and owners, vitally concerned in the present trend of State legislation, are banding together in many States in defense of their interests, and their association officers are beginning to correlate their efforts on a nation-wide scale.

Economists and business associations, such as the United States Chamber of Commerce, the National Industrial Conference Board, the National Tax Association, and others, are striving to find answers to those questions of cost of motor transport, public investment in highways, taxation of vehicles and the use to which the resulting tax funds are put, and the like, which will show accurately the true position of the motor vehicle in our economic structure and the facts by which its right to exist and develop should be judged.

From all these groups of serious and capable workers, convinced of the value of the motor vehicle to modern society and its present desperate need for fairness and intelligence in legislation, may soon be expected to arise contributions of the greatest value to the ultimate solution of the problem of motor vehicle regulation.

PROPOSED LEGISLATION

Despite the large volume of motor vehicle legislation already on the statute books, the wave of lawmaking in the States has far from spent itself. Early in 1933 the legislatures of 43 States are scheduled to hold their regular biennial sessions and, unless all signs fail, most, if not all of them, will consider bills which can hardly be expected to improve the existing complicated situation. The drive for increased tax revenue alone will be used to justify heavier burdens on motor vehicles, and particularly on the commercial types, notwithstanding the fact that returns during the first half of 1932 show conclusively that motor vehicle taxes have already entered a period of diminishing returns.

In the national field the United States Congress, during the coming session, will resume consideration of Federal regulation of motor vehicles in interstate commerce, in which it has been engaged ever since the second session of the Sixty-eighth Congress (1925–26). The first bills, proposing comprehensive control by the Interstate Commerce Commission of motor busses and motor trucks engaged in interstate commerce, were introduced by the respective chairmen of the Senate and House Committees on Interstate Commerce. These bills in modified form have been introduced at each succeeding session of Congress by the committee chairmen. The closest approach to enactment came in March, 1930, in the second session of the Seventy-first Congress, when the House passed its committee bill to regulate busses and reported it to the Senate, which left it as unfinished business on adjournment. At the next session this bill was recommitted, and died with the adjournment of the Seventy-first Congress. Committee bills in both Senate and House were again revised and introduced in the first session of the Seventy-second Congress, and much valuable testimony was taken upon the Senate measure,[24] but neither bill was reported out. Numerous other regulatory measures have also been introduced in both Houses of Congress. The present committee bills meet certain recommendations of the Interstate Commerce Commission, and differ from previous bills in that they separate holding-company provisions from the main bills.

Recommendations for Federal legislation were made in a report of the Interstate Commerce Commission on the subject of Coordination of Motor Transportation, issued this year.[25] This report contained a summary of evidence presented by many witnesses at a nation-wide series of hearings held in 1930 and 1931. In it the commission recommended Federal legislation on the following points: Regulation by the Interstate Commerce Commission of all motor carriers for hire engaged in transportation of persons or property on the public highways in interstate commerce; requirement of a permit to operate, and liability insurance; standard records of operations; authorization of through rates between common carriers by motor truck and common carriers by rail and by water; creation of joint boards composed of members of State regulatory bodies to cooperate with the commission in administering regulations; and the conducting of an impartial and authoritative investigation by Congress as to "whether and to what extent motor, water, and air carriers operating in competition with the railroads are receiving direct or indirect Government aid amounting, in effect, to a subsidy; and, if so, what steps, if any, are necessary to correct this situation, with a view to placing competition on a just and equitable basis; and that such investigation, if instituted, be extended to cover also the question of whether it is desirable in the public interest that regulations affecting public safety and convenience in the operation of motor carriers be made uniform throughout the country, and if so, how such uniformity may best be brought about."

This same report also recommends that "railroads, whether steam or electric, and water carriers, subject to the act, should be specifically authorized to engage in the transportation of both persons and property by motor vehicles in interstate commerce over the public highways and that thereafter such service, when directly engaged in by any such rail or water carrier, should be subject to the provisions of the interstate commerce act and legislation supplemental thereto."

In the opinion of the commission, "For the present no requirements should be made regarding the qualifications of drivers, hours of service of employees, and the size, length, weight of load, and speed of motor vehicles operating for hire on the public highways in interstate commerce." On this point the commission adds, "In order to obtain desirable uniformity in such regulations so far as they affect interstate commerce it may eventually become necessary for Congress to occupy this field."

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[23] Here may be cited an interesting phrase from the opinion of the United States Supreme Court in the Texas case previously referred to: "To make scientific precision a criterion of constitutional power would be to subject the State to an intolerable supervision hostile to the basic principles of our Government."

[24] Hearings before the Committee on Interstate Commerce, U. S. Senate, on Senate bill 2793, Feb. 1–8, Mar. 3–15, 1932.

[25] Report of the Interstate Commerce Commission on Docket No. 23400, decided Apr. 6, 1932.

POSSIBLE SOLUTIONS TO THE PROBLEM

We have seen how the motor vehicle within the span of a single generation has created a large number of unprecedented problems, and how the attempts to solve them thus far have been characterized by general lack of uniformity, with resultant costly inefficiency and waste. Now we turn our thoughts toward the future in an effort to indicate the general lines of possible action which may lead to more satisfactory solutions of our problems.

The first question which arises is that of jurisdiction. It must be said that the American Federal system of government, with police powers vested in 48 States, has made inevitable much of the nonuniformity presented by motor vehicle laws. In the absence of a centralized authority, and of an enlightened public opinion on these specialized subjects, consistency in legislation could hardly be expected. Yet, given the tremendous size of the field in which control is to be exerted—26,000,000 vehicles, operating over 3,000,000 miles of road, in 3,000,000 square miles of territory—Federal legislation over all phases of the problem is clearly impracticable. Such matters as registration, licensing, traffic regulations, and the like, are most efficiently handled by State units. On other points, however, such, for example, as the interstate operation of for-hire vehicles, the idea has been advanced that the Federal Government might find a proper and useful field of action.

With this general observation in mind, let us rearrange the elements of the problem as previously listed and indicate what now seem to be the most promising angles of approach in the search for amelioration of existing conditions. We may separate our elements into two main groups: Those over which the individual States seem best fitted to retain exclusive powers and those over which the Federal Government might take control if need for such action were definitely demonstrated.

REGULATIONS PROPERLY UNDER STATE CONTROL

Elements of regulation properly reserved to the individual States are outlined in the following paragraphs, together with the channels through which the efforts toward greater uniformity and rationalization of State laws will probably operate.

Registration.—The registration of vehicles and the issuance of identifying license plates are already practiced in all States. Although this phase of the problem is the least in need of remedial action, except, perhaps, as regards a proper type of administrative organization, the Uniform Vehicle Code of the National Conference on Street and Highway Safety offers useful suggestions for complete uniformity on these points. Act I of the code is titled as follows:

Uniform motor vehicle registration act.—An act to require the registration of motor vehicles, trailers, and semitrailers and to require the payment of fees thereupon and to require report to be made of any accident involving a vehicle and to impose certain duties and obligations upon the owners of motor vehicles rented without drivers, and to prevent the taking, transfer of, or injury to any vehicle without the consent of the owner; to provide for the office of vehicle commissioner and a department of motor vehicles and their powers and duties hereunder; to regulate court procedure in certain civil actions arising under this act; to provide penalties for violations of this act and to make uniform the law relating to the subject matter of this act.

In 10 States the registration laws are in substantial harmony with this act.

Antitheft legislation.—Protection for owners in the way of antitheft legislation is embodied in Act II of the Uniform Vehicle Code, the title of which reads as follows:

Uniform motor vehicle antitheft act.—An act to require certificates of title for registered motor vehicles; to facilitate the recovery of stolen or unlawfully taken motor vehicles; to provide for the licensing of dealers in used motor vehicles, trailers, or semitrailers; to prescribe the powers and duties of the vehicle department; to impose certain fees to carry out the purpose of this act; to impose penalties for violations of this act and to make uniform the law relating to the subject matter of this act.

Seven States and the District of Columbia have laws substantially in harmony with this act, while 15 others require some form of certificate of title.

Drivers' licenses.—The basically important requirement for the licensing of operators and chauffeurs is covered in Act III of the code, titled as follows:

Uniform motor vehicle operators' and chauffeurs' license act.—An act relating to the licensing of motor vehicle operators and chauffeurs and to the liability of certain persons for negligence in the operation of motor vehicles on the public highways and to make uniform the law relating thereto.

Eleven States have laws in substantial harmony with the code, while 16 others (including the District of Columbia) have some form of operator's license requirement.

Traffic regulation.—The regulation of traffic on the highways is given needed uniformity by Act IV of the code, under the following title:

Uniform act regulating traffic on highways.—An act regulating traffic on highways and providing for traffic signs and signals and defining the power of local authorities to enact or enforce ordinances, rules, or regulations in regard to matters embraced within the provisions of this act and to provide for the enforcement of this act and the disposition of fines and forfeitures collected hereunder and to make uniform the law relating to the subject matter of this act.

Some of the matters which it covers are as follows: Traffic signs and markers, accidents, reckless driving, speed restrictions, overtaking and passing and other rules of the road, right of way, pedestrians' rights, street cars and safety zones, and stopping, standing and parking, etc. Some of the provisions of this act, it should be noted, particularly in Article XVI, covering length of vehicles, have recently been held in abeyance pending further engineering research. Closely related to this act are two other valuable documents of the National Conference on Street and Highway Safety—a Model Municipal Traffic Ordinance and a Manual on Street Traffic Signs, Signals, and Markings. Eighteen States have laws substantially in harmony with Act IV of the code.

The value of this uniform vehicle code in standardizing State practices on the essential safety provisions above outlined can not be overemphasized. It represents a truly nation-wide approach to the general problem, and expresses the mature views of a host of competent authorities. It has been indorsed by the National Conference of Commissioners on Uniform State Laws and by the American Bar Association. Promotional activities in behalf of the code, looking toward its ultimate adoption by all the States, are in charge of the National Conference on Street and Highway Safety, 1615 H Street NW., Washington, D. C.

Financial responsibility.—The demand for financial responsibility on the part of owners and operators of motor vehicles, which in a comparatively short time has brought about enactment of legislation of this character in 19 States, may be expected to increase rapidly and the type of law briefly outlined on page 156 appears to offer the best hope of uniform action by the several States.

Physical characteristics of vehicles.—The wide disparity among present State regulations and the recent tendency toward further drastic restrictions in size and weight would seem to make uniform action by the individual States themselves almost hopeless. What are proper limits to the length, width, and height of vehicles or combinations of vehicles? Factors to be considered are the width of highways, radii of curves, clearance of overhead structures, traffic conditions, and other physical limitations to the passage of the larger vehicles, i. e., busses and trucks. How heavy may a road vehicle be? Here the chief considerations are the strength of the highway pavement and the load limitations of bridges. What should the equipment of a vehicle be? Under this heading may be classified tires, lights, brakes, horns, mirrors, windshield wipers, safety glass, connections between articulated vehicles, and the like. On some of these points Act IV of the Uniform Vehicle Code offers comprehensive suggestions; on others, new or revised recommendations are needed. The question of tires, particularly, needs careful consideration as the major factor in the effect of traffic on the highway pavement; the pneumatic and balloon tires have completely altered the effect of heavy vehicles on the road surface, and are now in practically universal use,[26] yet many State laws still appear to be based upon experience with solid tires. Recent development of the superballoon tire constitutes a new subject for study.

These are questions for engineers to answer. That they can be answered is not to be doubted, for highway construction practices have now progressed to a point where definite recommendations regarding the size, weight, and equipment of vehicles can be made on at least a "minimum-maximum" basis—that is to say, basic uniform limits below which no individual State need go in fixing the physical characteristics of vehicles using its highways, thus insuring to manufacturers, operators, and shippers a standard which they know will be recognized everywhere, but above which certain States may set higher limits for the benefit of their own vehicle users if they so desire. As already set forth, important groups are studying these problems and their findings will be of the greatest value. But the arrival at general agreement is a long process; even more distant is the enactment of such agreements into State laws. If early improvement is to be effected in the existing irrational situation, the Federal Government may be the only agency which can lead the States definitely in the direction of uniformity by exerting its powers over interstate commerce. Thus the individual State, while, in fact, retaining control over the physical characteristics of most of the vehicles using its highways, would be strongly impelled toward a uniform standard in fairness to these vehicles if the Federal Government should fix minimum-maximum dimensions for vehicles engaged in interstate commerce, or establish such standards as a condition for participation in Federal highway aid.

Business characteristics.—Unification of State regulation of vehicles according to their business characteristics appears to be particularly difficult. While the primary distinction between an owner-operated and a for-hire vehicle is well established, and while the common carrier constitutes a well-defined type of operation in most States, the contract carrier presents real difficulties in legislation because of the contractual and nonpublic nature of his service and his essential flexibility of operation. On purely intrastate operations the individual State has exclusive jurisdiction, and any improvement in the status of for-hire operations within any particular State will likely depend upon the degree of success attained in convincing the public and the legislative assembly of the value of motor-transport service. On interstate operations, where no intrastate service is performed, the Federal Government has the power to legislate if it so desires, and the numerous bills now pending in Congress indicate that some action along this line may ultimately be taken.

Taxation.—The taxation of motor vehicles, which in the final analysis sums up and expresses concretely all the costly inconsistencies in existing State legislation, is the one phase of the entire problem of regulation which is of most immediate concern to every motor-vehicle owner, and to all those who use motor transport directly or indirectly.

A total tax bill of more than $1,000,000,000 annually in the United States proves the vital importance of this question. During the coming months there will be staged in practically all States a battle between those seeking increased revenue through higher taxes on motor vehicles and those opposing further taxation as endangering the entire fabric of highway transportation, private and public alike. Unfortunately, little of practical value can be suggested as a remedy for the present complex tax situation. The power of the various taxing jurisdictions, whether State, county, or municipal, is not to be questioned, and the tax-levying authorities are answerable for their policies only to their electors. The final solution of these perplexing problems will depend upon an enlightened public attitude as to the value of motor transportation in all its phases, and to the effect of placing undue burdens upon the vehicle and thereby curtailing its usefulness. Here, too, engineers and economists, working to determine the truth about highway costs and usage, and to establish fair and accurate bases for registration fees, gasoline taxes, business taxes, and the like, will be able to guide public opinion by their recommendations.[27]

Reciprocal privileges.—Uniformity of regulations in the several States would go a long way toward simplifying the problem of reciprocal privileges in the movement of motor vehicles across State lines, but would not solve it entirely. A standard registration system, adequate drivers' licenses and financial responsibility, uniform requirements as to physical characteristics of vehicles, and universally recognized rules of the road, would actually afford much greater freedom of movement to vehicles and safety to the public than now exists; but would not answer the question any State might raise: "What are these foreign vehicles paying for the use of our roads?" The principle of reciprocity here encounters the right of taxation, and the only practicable answer is in the nature of a compromise. Privately owned and operated vehicles, including both passenger cars and trucks, already enjoy a favored position as to reciprocal privileges under the laws of numerous States, and the trend toward fuller reciprocity for these vehicles is becoming fairly well established, so that in time free movement everywhere may be possible for any private vehicle properly registered and tax-paid in its State of origin. As regards for-hire vehicles, an equitable apportionment of their privilege or business taxes, in lieu of license fees, among the States they enter or traverse, based on the extent of their operations in each State (e. g., ton-miles or revenue), would eliminate the onerous multiple taxation to which they are now frequently subject. If these for-hire vehicles were to be divided into classes as to regularity or irregularity of service, the problem of control would be further simplified by eliminating from consideration those casual for-hire operators whose use of the highways in any particular State other than the State of registration is sporadic, allowing them the same full reciprocity status as privately operated vehicles.

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[26] Of the 50,270,136 tires shipped by American tire manufacturers in 1931 only 209,445 tires, or less than one-half of 1 per cent, were of the solid and cushion types, according to the Rubber Manufacturers' Association (Inc.).

[27] Research by the Bureau of Public Roads has resulted in a number of reports bearing on this problem, and complete lists of references will be supplied upon request. The following are thought to be particularly pertinent:

Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, Public Roads, Vol. 7, No. 2, April, 1926.

Mechanics of Progressive Cracking in Concrete Pavements, by H. M. Westergaard, Public Roads, Vol. 10, No. 4, June, 1929.

Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness, by J. A. Buchanan and J. W. Reid, Public Roads, Vol. 7, No. 4, June, 1926.

Interrelated Effects of Load, Speed, Tires, and Road Roughness on Motor Truck Impact, by J. A. Buchanan, Public Roads, Vol. 11, No. 7, September, 1930.

Impact Reactions Developed by a Modern Motor Bus, by J. A. Buchanan, Public Roads, Vol. 12, No. 2, April, 1931.

Road Impact Produced by Heavy Motor Bus, by J. A. Buchanan, Public Roads, Vol. 13, No. 9, November, 1932.

Report of a Survey of Transportation on the State Highway System of Ohio, 1927.

Report of a Survey of Traffic on the Federal-Aid Highway Systems of 11 Western States, 1930.

Domestic Commerce Series No. 66, Motor Truck Freight Transportation, published by the Bureau of Foreign and Domestic Commerce in cooperation with the Bureau of Public Roads, 1932.

Statistical tables issued annually by the Bureau of Public Roads, including: Motor vehicle registrations and fees, Gasoline taxes, State highways built during year, State highways existing at end of year, State highway income during year, State highway expenditures during year, Local roads built during year, Local roads existing at end of year, Local highway income during year, Local highway expenditures during year.

It should be said that the gasoline tax, which amounted last year to almost exactly half of all motor vehicle taxes collected, obviously represents a return to each State from the vehicles using its highways and thus provides a material compensation to each State for the use of its highways by foreign vehicles, which may be actually increased by liberal reciprocity measures. Here again is a field for exclusive State action, and one in which public opinion and the enlightened self-interest of State authorities must be depended upon for more liberal regulations.

Enforcement.—Since the major portion of the whole problem of motor vehicle regulation appears destined to remain within the control of the individual States, the question of enforcement must remain primarily a State matter. Indications point to the eventual establishment of some sort of highway patrol, especially charged with enforcement of the motor vehicle laws on the public roads, in all States where the traffic problem is sufficiently important to warrant it. Eventually that may well mean every State in the Union.

Regulation of certain special aspects of for-hire operations will also logically rest in the several States when such operations are of intrastate character. Hours of labor, rates, practices, and services of for-hire vehicles, certificates of convenience and necessity for common-carrier operations, insurance or other special financial responsibility requirements, come within this category.

On many of these questions which lie within the exclusive jurisdiction of the individual States there is a pronounced lack of definite data to serve as a basis for wise legislation. The Uniform Vehicle Code, which is a prototype of the kind of action needed, covers but a portion of the whole field of regulation. The remaining elements, which the code does not touch upon, depend for their solution upon the results of special study by each State of its own motor-vehicle problems. Immediate opportunity for an intelligent approach to these questions is presented by the State legislative assemblies which convene in the near future. Given the complexity of the existing situation, and its importance to the general welfare of the Nation, action of the new legislatures might profitably be diverted from creation of further nonuniform statutes, and confined to the creation of commissions of investigation with specific instructions to analyze the problem of motor vehicle regulation within the State and, in addition, to confer with commissions of other States working along similar lines. Here would appear to be the most fruitful field of endeavor now open to the States themselves.

THE QUESTION OF FEDERAL CONTROL

On certain elements of the general problem the suggestion has been made from time to time that Federal control is the only means of achieving rational and uniform regulations on a nation-wide scale. Such control, however, has never been suggested as all-inclusive over all types of vehicles, but rather as limited to interstate for-hire operations.

There is no question as to the authority of the Federal Government to enter the field of interstate motor vehicle regulation if it so desires. As stated in the Interstate Commerce Commission Report on Docket 23400, page 382, "It is well settled that the power over commerce among the States conferred upon Congress by the Constitution is complete in itself, extends incidentally to every instrument and agent by which such commerce is carried on, may be exerted to its utmost extent over every part of such commerce, and is subject to no limitations save such as are prescribed in the Constitution." This Federal power has already been exercised over interstate commerce by railroads, pipe lines, telegraph and telephone lines, and by air, and an agency of the Federal Government, the Steamboat Inspection Service of the Department of Commerce, passes upon the seaworthiness of all vessels in water-borne commerce. Even communication by radio is supervised by a Federal body. The field of highway transportation alone has not yet been entered by the National Government.

The question of Federal control, therefore, is not one of legislative powers, but rather one of practicability. Here arise matters of policy to which no final answers can yet be given, and the present discussion accordingly must be confined to a brief statement of some of the considerations involved.

In a broad sense, Federal control over interstate motor transportation, as currently suggested, may be divided into two parts: The regulating of operations of interstate for-hire vehicles according to their business characteristics, and the fixing of minimum-maximum limits to the dimensions, weight, and equipment of vehicles used in such operations. Present legislation in Congress treats of only the first of these.

Decisions on the proper policy for business regulation depend upon the character of the various classes of vehicles to be controlled. For example, interstate common carriers of persons (busses) present a simpler problem than other classes of carriers, and it has been suggested that a competent Federal authority could administer them without great difficulty and possibly with some improvement over the existing system of individual State administration. Such control would logically include issuance of interstate certificates of convenience and necessity, requirements with respect to service, hours of labor of drivers, accounting records and reports, rates or fares, routes, and public liability. The fact that every State of the Union except one now regulates the operation of common carriers of persons constitutes a firm basis for interstate regulation by a Federal body.

The case of the for-hire carrier of property is not so easy to decide. It has been explained that there can hardly be more than 500,000 trucks now engaged in this class of service, and that the number operating interstate is certainly very much less. But, in any case, it has been pointed out, even 100,000 trucks, owned by many thousands of individual companies or persons, would present a very difficult problem in centralized administration. No entirely satisfactory plan for handling these operators on a Federal basis seems yet to have been advanced, and current opinion appears to regard any control of them by a Federal body to be, at the outset at least, largely of an experimental character. It is along this line of reasoning, apparently, that pending legislation in Congress sets up a simple permit system for carriers of property, both of the common and contract type, designed chiefly to obtain useful data on the scope of the problem, which will permit of possible future recommendations as to more specific means of regulation. From the point of view of the public administrator this method of ascertaining the scope of the problem might prove of value, although there have been some predictions that the plan may be found cumbersome in administration and difficult of enforcement. From the standpoint of the operator and shipper, frank views have been expressed that it will mean an additional burden of expense without compensating advantages. In this divergence of opinion the lack of pertinent data remains the chief obstacle to an equitable decision.

The fixing of size and weight specifications for interstate for-hire vehicles as a proper activity of the Federal Government has also been given consideration, and the view is held in some influential quarters that action on this point would prove a helpful contribution to the task of standardizing this phase of the general problem. Such standardization, it has been held, involving as it does important considerations of public safety and highway construction practices, could hardly fail to benefit vehicle manufacturers, shippers, carriers, and the general public. On the other hand, action of this kind by the Federal Government has been pronounced an invasion of the rights of the individual States.

Regardless of the ultimate decision on this knotty point, a brief discussion of the question of Federal specifications may be of interest here. The general subject of physical characteristics of vehicles, and the minimum-maximum principle for their standardization, have already been explained. It is sufficient, therefore, to add a word about the actual method of fixing such minimum-maximum limits on a national basis.

It may be said that specifications for length, height, width, and equipment of vehicles which may be safely operated on the major interstate highways in every part of the Nation can be fixed to-day by competent engineers. Likewise, a formula for the weight of vehicles, taking into account the factors of strength of highway pavement and bridges, can be framed which will meet the varying requirements of all sections of the country. The highway pavement can be protected by a limitation on the axle load without regard to the total gross load on a vehicle or combination of vehicles. Bridges can be protected by a flexible limitation on the gross load. For this purpose the use of formulas giving the gross load as a function of the distance between the first and last axles of a vehicle or combination of vehicles has been suggested, and such formulas have been adopted in seven States.

Whatever may be the form of State or Federal limitations, provision will always be needed for the proper authorities to issue special permits for the movement of exceptional loads and sizes of vehicles.

CONCLUSION

What is needed most of all, in consideration of the problem of regulating motor vehicles, is more factual data. Not opinions, but facts, should govern the course of future legislation. Broad and truly impartial investigations of the whole problem must be made from diverse points of view. From engineers must come facts as to the cost of construction and maintenance of the various types of highways designed to meet various traffic needs, and the effect upon the roads of the physical characteristics of vehicles; from economists, facts as to the true place of motor transport in the general transportation structure, and the fair charges to be borne by the various types of vehicle in compensation for the road facilities provided for them; from safety experts, further advice on the uniform control of traffic and the reduction of accidents; from officials of Government, facts on standardized regulation, adequate protection for the public, reciprocal privileges among the States, and effective enforcement. In fact-finding investigations of this kind a complete delineation of the elements of the problem such as has been only suggested in this article would appear essential to clear and coordinated results. Agreement upon definitions and terminology alone would not be the least important part of these studies.

On those elements of the problem which have been suggested as falling within the jurisdiction of the individual States the goal of greater uniformity is to be attained through continuation of the educational and cooperative efforts already under way. On those matters which concern interstate commerce a new policy may be necessary. The cost and inefficiency of present conditions must be weighed against the potential difficulties of a new and hitherto untried system of control. The final answer must depend upon the intelligent collaboration of all those interested in the fair and rational development of that indispensable tool of modern society, the motor vehicle.

APPENDIX A

State legislation on certain aspects of motor vehicle regulation

[From "Digest of Regulations Governing the Operation of Motor Vehicles throughout the United States and Provinces of Ontario and Quebec, Canada, 1931-32," compiled by E. Austin Baughman, commissioner of motor vehicles, State of Maryland, Baltimore, Md.]

State	"Stolen car" department or files	Certificate of title	Operator license	Chauffeur license	Minimum age for operators	Examination for operator license	Compulsory inspection of vehicles	State patrol force or inspectors	Maximum speed permitted private passenger cars in open country
					Years				*Miles per hour*
Alabama				x	16				45.
Arizona	x	x	x	x	16	l		x	35.
Arkansas				x				x	35.
California	x	x	x	x	16	ldv		x	45.
Colorado	x	x	x	x	15	ldv			rp.
Connecticut			x	x	16	ldv	(1)	x	rp.
Delaware	x	x	x	x	16	ldv	x	x	40.
Florida	x	x		x	14				45.
Georgia	x			x	16				40.
Idaho	x	x		x			x	x	35.
Illinois				x	15				45.
Indiana	x	x	x	x	16		(2)	x	rp.
Iowa			x	x	15	l		x	rp.
Kansas			x	x	16	ld		x	rp.
Kentucky				x	16				rp (40).
Louisiana	x			x				x	45.
Maine			x	x	15			x	rp (35).
Maryland	x	x	x	x	16	ldv	x	x	40.
Massachusetts	x		x		16	ldv	x	x	pr (30.)

[1] Busses only.

[2] Cities only.

State legislation on certain aspects of motor vehicle regulation—Con.

State	"Stolen car" department or files	Certificate of title	Operator license	Chauffeur license	Minimum age for operators	Examination for operator license	Compulsory inspection of vehicles	State patrol force or inspectors	Maximum speed permitted private passenger cars in open country
					Years				*Miles per hour*
Michigan	x	x	x	x	14	.....	...	x	rp.
Minnesota	x	.....	.....	x	....	.....	.....	...	45.
Mississippi	x	.....	.....	.....	.....	.....	.....	.....	30.
Missouri	x	x	.....	x	16	.....	.....	...	rp (25).
Montana	x	x	.....	x	.....	.....	.....	x	rp.
Nebraska	.....	.....	x	..	16	.....	x	x	45.
Nevada	x	.....	x	x	15	.....	.....	..	rp.
New Hampshire	.....	.....	x	x	16	ldv	x	x	35.
New Jersey	.....	.....	x	.....	17	ldv	.....	x	40.
New Mexico	x	x	.....	.....	14	.....	.....	.....	45.
New York	x	.....	x	x	18	dv	x	x	40.
North Carolina	x	x	.....	.....	16	.....	.....	.....	45.
North Dakota	x	x	.....	.....	.....	.....	.....	x	50.
Ohio	.....	.....	.....	x	.....	.....	.....	.....	45.
Oklahoma	x	x	.....	.....	.....	.....	.....	.....	45.
Oregon	x	x	x	x	16	ldv	.....	x	45.
Pennsylvania	x	x	x	.....	16	ldv	x	x	40
Rhode Island	..	.....	x	.....	16	lv	.....	x	rp (35).
South Carolina	x	.....	x	.....	12	ld	...	x	45.
South Dakota	x	x	.....	.....	15	.....	.....	.....	40.
Tennessee	x	.....	.....	.....	.....	.....	.....	x	ns.
Texas	.....	.....	.....	x	.....	.....	.....	.....	45.
Utah	.....	x	.....	.....	16	.	...	.....	45.
Vermont	x	.....	x	.....	18	ldv	.....	x	ns.
Virginia	x	x	x	x	.....	(³)		x	45.
Washington			x	...	15	...	...	x	40
West Virginia	x	x	x	x	15	ldv	...	x	45.
Wisconsin	x	x	x		16			x	ns.
Wyoming				...	15	.		.	rp (35).
District of Columbia	x	.	x	...	16	ldv		...	30[4]
Total	32	23	26	30	.....	.	10	30	...

[3] Operator license law adopted in Virginia in 1932, examination of new applicant (minimum age, 16 years) obligatory after July 1, 1933.

[4] On certain streets in the District of Columbia only; speed limit elsewhere, 22 miles per hour.

NOTE.—The following abbreviations are used, either singly or in combination:
x—Legislative and/or administrative action.
d—Driving test.
l—Written or oral test on motor vehicle laws.
v—Test of vision.
rp—Not greater than is "reasonable and proper"; figure in parentheses indicates limit beyond which higher speed is deemed prima facie evidence of improper driving.
ns—Not specified.

APPENDIX B

Maximum limitations imposed by States on size and weight of motor vehicles on highways outside of incorporated areas,[1] 1932

[ns—not specified; u—single unit; c—combination of units; s—solid tires; p—pneumatic tires; d—dual tires; pl—pay load; L—distance in feet between first and last axle, where weight is given by formula; trailer designations: ½—semitrailer only; 1—1 semitrailer or full trailer; 1½—1 semitrailer and 1 full trailer, 2—2 trailers, either type]

State	Over-all length	Over-all width	Over-all height	Gross weight on 1 axle	Gross weight of 4-wheel vehicle	Gross weight of 6-wheel vehicle	Maximum combination permitted and corresponding gross weight		Remarks
							Number of trailers	Gross weight of combination[2]	
	Feet	*Inches*	*Feet*	*Pounds*	*Pounds*	*Pounds*		*Pounds*	
Alabama	33u 46c	96	12	12,000	21,000	32,000	1½	32,000	
Arizona	33u 85c	96	14½	18,000	22,000	34,000	1½	90,000	
Arkansas	33u 85c	96	14½	[3] 14,820d	25,000+650L	25,000+650L	1½	25,000+650L	Solid tires not permitted after July 1, 1932.
California	33u 60c	96	13½	17,000	22,000	34,000	1½	68,000	
Colorado	33u [4] 60-85c	96s 100p	12½	20,000	30,000	40,000	1½	80,000	
Connecticut	40	102	ns	ns	26,000s 32,000p	26,000s 40,000p	1	26,000s 40,000p	
Delaware	33u 60c	96	12½	16,000s 18,000p	22,000s 26,000p	22,000s [5] 38,000p	1½	48,000s 59,000p	
Florida	35u 45c	[6] 84	12	ns	16,000 20,000	16,000 20,000	1½ 1½	24,000 40,000	Private carriers, permit carriers, and certificated private contract carriers. Certificated common carriers.
Georgia	30u [7] 85c	96	12½	17,600	22,000	39,600	ns	ns	Truck tractor, 4-wheel semi and 6-wheel full trailer maximum gross, 83,600 pounds.
Idaho	33u 85c	96	14½	16,000	24,000	40,000	1½	40,000	
Illinois	35u [8] 40c	96	ns	16,000	24,000	40,000	ns	ns	Maximum gross weight of trailer or semitrailer, 32,000 pounds. After Jan. 1, 1933, pneumatic tires required on all busses, and on all trucks operating at more than 10 miles per hour.
Indiana	33u 40c	96	12	12,800s 16,000p	600 (L+40)	600 (L+40)	1½	600 (L+40)	
Iowa	30u 45c	96	12	14,000s 16,000p	24,000+450L	24,000+450L	ns	24,000+450L	
Kansas	35u 52c	96	13	16,000 18,500	24,000 28,000	34,000 34,000	2 2	102,000 102,000	On less than 8-inch dual tire equipment. (Solid tires prohibited, except for farm machinery and produce-hauling vehicles.) On 8-inch duals. (Solid tires prohibited, except for farm machinery and produce-hauling vehicles.)
Kentucky	26½u 30c	96	11½	ns	18,000	18,000	½	18,000	Full trailers prohibited.
Louisiana	33u [9] 55c	96	12½	ns	7,000pl	7,000pl	2 1	24,000pl 14,000pl	Until Jan. 1, 1934. After Jan. 1, 1934.

[1] An abstract of State laws, including legislation passed in 1932. Figures for the District of Columbia are also included. Complete information, including changes which may be made subsequent to the publication of this table, may be obtained from the State authorities. It should be noted that many States exempt farm machinery and produce-hauling vehicles from the size and weight restrictions listed in this table; and that special permits may be issued for the movement of excess sizes and loads.

[2] In all cases gross weight includes motor unit (truck or tractor) plus trailer equipment indicated.

[3] Varies with tire size. Axle load can not be greater than 14,820 pounds.

[4] In mountains, 60 feet, elsewhere, 85 feet.

[5] With power brakes on both rear axles; without power brakes, 36,000 pounds.

[6] Tread width center to center. For-hire vehicles, 96 inches over all.

[7] Motor carriers: Single unit, 35 feet; combination, 45 feet.

[8] Effective Jan. 1, 1933. Tractor-semitrailer combination restricted to same over-all length as single unit (35 feet).

[9] After Jan. 1, 1934, 45 feet.

Maximum limitations imposed by States on size and weight of motor vehicles on highways outside of incorporated areas, 1932—Con.

[ns—not specified; u—single unit; c—combination of units; s—solid tires; p—pneumatic tires; d—dual tires; pl—pay load; L—distance in feet between first and last axle, where weight is given by formula; trailer designations: ½—semitrailer only; 1—1 semitrailer or full trailer; 1½—1 semitrailer and 1 full trailer; 2—2 trailers, either type]

State	Over-all length	Over-all width	Over-all height	Gross weight on 1 axle	Gross weight of 4-wheel vehicle	Gross weight of 6-wheel vehicle	Maximum combination permitted and corresponding gross weight		Remarks
							Number of trailers	Gross weight of combination	
	Feet	*Inches*	*Feet*	*Pounds*	*Pounds*	*Pounds*		*Pounds*	
Maine	36u 62c	96	12½	13,500	20,000s 24,000p	27,000	1	27,000	Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross 80,000 pounds. No combination including more than 1 semitrailer or full trailer may be operated at more than 10 miles per hour.
Maryland	ns	93	ns	ns	25,000	40,000	ns	ns	
Massachusetts	[10] 28u ns (c)	96s 102p	ns	ns	28,000s 30,000p	28,000s 30,000p	1	56,000s 60,000p	
Michigan	40u 60c	96	14	1,700–18,000	-------	------------	2	---------------	Axle and gross loads permitted vary with axle spacing, seasons, and road types.
Minnesota	35u 63c	96	12½	14,400s 18,000p	28,800s 36,000p	33,600s 42,000p	2½	ns	Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross: Solids, 81,200 pounds; pneumatics 102,000 pounds.
Mississippi	33u 50c	96	12½	12,000	22,000	22,000	ns	30,000	Maximum pay load permitted, 7 tons.
Missouri	33u 40c	96	12½	16,000	24,600	24,000	1	48,000	
Montana	33u 60c	96	14½	[11] 16,800 and 13,000	24,000	34,000	1½	68,000	
Nebraska	35u 50c	96	12	16,000	16,000pl	16,000pl	1½	32,000pl	
Nevada	ns	96	ns	ns	25,000	38,000	ns	ns	Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross, 76,000 pounds.
New Hampshire	30u 85c	96	ns	15,000	20,000	20,000	ns	ns	Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross, 60,000 pounds.
New Jersey	33u 66c	96	12½	([12])	30,000	30,000	1	60,000	
New Mexico	33u 85c	96	14	18,000	600 (L+40)	600 (L+40)	1	600 (L+40)	Additional trailers by special permit.
New York	33u 65c	96s 106p	13	17,920s 22,400p	28,800s 36,000p	35,000s 44,000p	2	750 (L+40)	
North Carolina	33u 55c	90	12	ns	20,000	20,000	1½	60,000	
North Dakota	35u 70c	96	14½	16,000	32,000	48,000	1	64,000	
Ohio	35u 85c	96	12½	16,000s 18,000p	20,000s 24,000p	20,000s 24,000p	ns	ns	6-wheel truck and two 4-wheel full trailers, maximum gross: Solids, 60,000 pounds; pneumatics, 72,000 pounds.
Oklahoma	ns ns	90	ns	16,000	20,000	20,000	ns	ns	Truck tractor-semitrailer combination regarded as single vehicle; truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross, 40,000 pounds.
Oregon	34u [13] 50c	96	11	16,000 [14] 17,000	600 (L+40)	600 (L+40)	ns	600 (L+40)	Solid tires not permitted except in municipalities or within 3 miles thereof.
Pennsylvania	33u 70c	96	14½	18,000	26,000	36,000	1½	65,000	
Rhode Island	85c	102	12½	22,400	28,000s 32,000p	40,000	2	120,000	
South Carolina	33u 50c	90	12½	[15] 15,000 and 10,000	25,000	25,000	ns	40,000	
South Dakota	50c	96	12½	16,000	20,000	20,000	1	40,000	
Tennessee	35u 45c	96	12	18,000	20,000	20,000	1½	60,000	
Texas	35u 45c	96	12½	ns	7,000pl	7,000pl	1	7,000pl	14,000 pounds pay load permitted when being transported to or from nearest common carrier loading point.
Utah	33u 85c	96	14	13,500s 18,000p	19,500s 26,000p	25,500s 34,000p	ns	45,000s 60,000p	
Vermont	ns	96	12	ns	16,000 20,000	16,000 20,000	1 1	16,000 20,000	Permitted on all highways. Permitted on State-aid roads. Permits required in all cases for gross loads above 20,000 pounds
Virginia	33u [16] 45c	96	12½	16,000	24,000	35,000	1	35,000	Combination including semi and full trailer, if registered prior to Mar. 26, 1932, may be operated until Apr. 1, 1934.
Washington	35u 85c	96	ns	[17] 12,000–18,500	24,000	34,000	1	60,000	
West Virginia	33u 85c	96	12	8,000–22,400s 16,000–22,400p	([18])	([18])	ns	([18])	
Wisconsin	33u 60c	96	ns	19,000 12,000	24,000 15,000	36,000 22,500	1½ 1½	96,000 60,000	Class A highways. Class B highways. Solid tires prohibited after July 1, 1934.
Wyoming	40u 85c	96	12½	18,000	30,000	30,000	ns	30,000	Permits required on all gross weights exceeding 30,000 pounds.
District of Columbia	33u 85c	96	12½	[19] 22,400s [19] 24,640p	28,000s 30,800p	[20] 36,000s [20] 39,600p	ns	ns	Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross: Solids, 72,000 pounds; pneumatics, 79,200 pounds. New registration of solid tired vehicles under special permit only.

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[10] 33 feet on principal routes, as designated by Department of Public Works; tractor-semitrailer, 40 feet.
[11] Varies with number of axles: 2 axles, 16,800 pounds; more than 2 axles, 13,000 pounds.
[12] Varies with tire equipment.
[13] Effective Jan. 1, 1933.
[14] On paved highways.
[15] Varies with number of axles: 2 axles or less, 15,000 pounds; more than 2 axles, 10,000 pounds.
[16] Exclusive of coupling.
[17] Varies with number and spacing of axles.
[18] Gross load determined by capacity of bridges by following formulas: H-20 design, gross load=1,330 (L+40); H-15 design, gross load=1,000 (L+40); H-10 design, gross load=670 (L+40).
[19] Limited to 14,000s and 15,400p for vehicles with 6 or more wheels, a tractor and semitrailer combination being considered as a single vehicle as regards weight limitations only.
[20] Tractor and semitrailer combination is considered as a single vehicle of 6 or more wheels as regards weight limitations only.

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ROAD PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the department and as the department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1925. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1926. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1930. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

DEPARTMENT BULLETINS

No. 136D . . Highway Bonds. 20 cents.

No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects. 5 cents.

No. 93MC . . Direct Production Costs of Broken Stone. 25 cents.

No. 109MC . . Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters. 10 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . . The results of Physical Tests of Road-Building Rock. 25 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They can not be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927.)

Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States. (1930.)

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS was printed in PUBLIC ROADS, vol. 13, No. 3, May 1932. Copies of this list may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

NOVEMBER 30, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2.368.7	$ 4,225,833.74	$ 2,091.020.77	45	161.5	22.3	183.8	$ 1.316,725.83	$ 658.362.89	25.2	27.7	52.9	$ 3.599.425.81	Alabama
Arizona	1.205.3	3,476.642.04	1.958.233.71	77	114.5	127.9	242.4	351,160.88	70.232.17	24.7	21.9	46.6	199,857.78	Arizona
Arkansas	1,957.9	2.815.622.27	1,304,878.41	76	73.8	38.4	112.2	822.908.45	411,454.18	23.3		23.3	1.821,894.43	Arkansas
California	2,448.2	7.598,702.42	2,805,910.77	69	155.0	46.2	201.2	2.154,334.90	203.326.98	27.3	5.0	32.3	208,732.23	California
Colorado	1.708.8	2,897,618.36	1,404,318.24	75	105.6	61.0	166.6	628,461.48	282,807.62	30.9		30.9	1.090,882.68	Colorado
Connecticut	289.8	4,291,246.27	1,633.337.97	66	35.3		35.3	223.488.92	111,744.45	6.2		6.2	222.621.88	Connecticut
Delaware	383.3	336.916.35	83.647.50	62	10.5	3.9	14.4	235.681.55	46.000.00	14.8	13.3	28.1	69.252.14	Delaware
Florida	660.2	5,003,601.52	2,364,040.75	55	131.0		131.0	602.870.65	301.435.30	17.8		17.8	1.583.388.84	Florida
Georgia	3,131.8	6,305,127.53	2,537.438.17	67	150.2	147.8	298.0	886.903.32	319.861.98	55.5	21.8	77.3	326.852.14	Georgia
Idaho	1.432.5	2,290,147.14	1,144.271.69	91	112.8	194.8	307.6	1,111.644.30	290.292.61	50.5	11.6	62.1	93.982.62	Idaho
Illinois	3,020.3	21,131,126.14	8,563.574.00	80	660.3	32.0	692.3	1.931,600.85	485,217.35	72.5		72.5	228.045.37	Illinois
Indiana	1.962.4	8,878.376.01	4,334.652.70	91	327.6	3.0	330.6	906.878.08	90.100.00	48.3		48.3	253,447.32	Indiana
Iowa	3,436.9	7,098.249.92	1.721.281.21	90	322.8	62.8	385.6	128,224.42	16.860.00	6.9	5.2	12.1	4,983.72	Iowa
Kansas	3.754.8	4,374,747.43	2,120,114.15	65	262.2	110.0	372.2	2,158.639.62	332.288.39	131.4	47.9	179.3	85.675.43	Kansas
Kentucky	1.842.3	5,059,193.80	1.877,219.57	60	228.5	127.3	355.8	528.952.24	160.270.00	26.1	14.9	41.0	193.825.49	Kentucky
Louisiana	1,609.9	6,470,517.34	2,333.980.42	50	63.1	18.7	82.1	1.074,100.16	499.869.51	8.4		8.4	2,427.94	Louisiana
Maine	799.1	1.556.289.58	584,864.64	87	59.6		59.6	834,836.12	124,460.31	21.3	2.2	23.5	10,636.63	Maine
Maryland	813.9	1,298,028.30	582,192.68	85	50.0	4.7	54.7	655.494.55	42.260.75	24.8	1.5	26.3	19,478.90	Maryland
Massachusetts	856.8	4.993,936.42	1.698,921.87	73	53.9	5.0	58.9	550.120.53	80,000.00	10.0		10.0	74.036.41	Massachusetts
Michigan	2,237.3	7.634,204.37	3,288.010.00	79	310.2	74.7	384.9	964.100.00	339.402.50	82.7	11.7	94.4	535.739.41	Michigan
Minnesota	4,220.7	6.587,773.02	814,130.67	97	157.2	216.7	373.9	237,800.71	200.00	13.0	.4	13.4	17.894.76	Minnesota
Mississippi	1,840.9	4.710,041.98	2.315,393.90	70	189.8	53.9	243.7	709.959.63	354.979.77	29.7	6.4	36.1	4.101.607.65	Mississippi
Missouri	3.105.3	4,044,679.12	1,372,095.25	84	116.4	45.8	162.2	918,084.10	214.119.71	63.9		63.9	382,820.33	Missouri
Montana	2,724.1	5,254.429.32	2.954,877.52	70	410.7	206.7	617.4	2,562,104.58	1,438,012.85	192.1	48.7	240.8	374,932.46	Montana
Nebraska	4.260.3	4.845.021.86	2,314,953.79	45	140.4	134.4	274.8	880.233.33	407.306.14	16.5	21.7	38.2	400.972.17	Nebraska
Nevada	1.347.1	1,543.571.62	920,884.95	83	29.7	115.0	144.7	342.911.03	29.582.33	1.1	33.3	34.4	62.892.19	Nevada
New Hampshire	427.1	833.618.11	369.739.95	79	27.6	3.5	31.1						217.519.61	New Hampshire
New Jersey	623.6	7.680,171.79	2,670,839.30	58	68.0	.5	68.5	178.205.74	79.176.08	1.5		1.5		New Jersey
New Mexico	2.235.8	2.295.709.15	1.127.403.76	57	136.6	55.8	192.4	771,450.11	319.347.34	40.5	37.3	77.8	433.454.38	New Mexico
New York	3.399.0	20,426.910.00	6.686.165.00	56	525.5	31.2	556.7	1.449.400.00	532,250.00	34.2	.2	34.4	691,018.55	New York
North Carolina	2,255.2	1,954,711.89	978.894.78	39	201.7	20.9	222.6	1,937.269.00	970,471.95	262.3	.4	262.7	2.143.491.76	North Carolina
North Dakota	5.132.4	4.357.006.67	1.644,938.72	67	414.7	456.6	871.3	1.318,469.39	760.619.70	62.7	249.5	312.2	85.566.59	North Dakota
Ohio	2.910.2	9.972.781.34	3.148,811.15	84	232.5	98.5	331.0	3,294.422.20	655,698.68	40.2	1.3	41.5	212.567.01	Ohio
Oklahoma	2.375.1	4.004.992.44	1,981.935.01	76	210.3	37.7	248.0	996.253.16	288,947.65	53.5	19.9	73.4	845.996.32	Oklahoma
Oregon	1.633.0	2,418.166.30	1.160.846.90	72	64.4	54.8	119.2	1.749,980.82	530.772.31	54.2	54.9	109.1	267.582.01	Oregon
Pennsylvania	3.145.1	8.282,860.11	3,184.438.09	67	320.3		320.3	4.168.655.37	503,526.92	137.7	14.1	151.8	571,828.96	Pennsylvania
Rhode Island	263.8	1,146.278.45	431,112.44	47	21.4	4.3	25.7	161.649.03	20.000.00	3.6		3.6	84.436.47	Rhode Island
South Carolina	1.990.4	3,236,640.92	1.270.692.73	53	179.6	112.4	292.0	102.258.53	34,200.00	4.9		4.9	152.353.34	South Carolina
South Dakota	3.974.4	5.150,976.59	2,402.618.68	76	434.6	276.3	710.9	66.290.85	32.486.02	9.9		9.9	66.211.12	South Dakota
Tennessee	1,697.8	4.006.434.89	2.002.920.23	26	128.9	42.0	170.9	530.141.83	264.350.06	21.7	7.5	29.2	1,257.607.29	Tennessee
Texas	7,774.4	15.802.284.52	6.853.093.83	66	726.4	402.1	1.128.5	6.328,284.97	1.924.478.99	298.7	168.7	467.4	395.531.96	Texas
Utah	1,211.3	1.638,247.58	861.120.06	84	102.2	83.2	185.4	669.488.14	190.182.40	32.1	28.0	60.1	248,754.16	Utah
Vermont	364.7	836.860.25	309.246.92	86	37.4		37.4	187.283.63	22.678.94	8.7		8.7	10.353.50	Vermont
Virginia	1.933.4	3,548.128.72	1,693.117.59	58	194.8	26.4	221.2	1.489.585.32	429.597.24	40.5	21.8	62.3	769.802.49	Virginia
Washington	1.294.5	2,530.944.20	982.812.10	69	96.8	6.3	103.1	845.859.62	279.000.00	33.7		33.7	229.857.85	Washington
West Virginia	913.0	3.315.196.60	1.282.638.44	54	112.4	8.4	120.8	386.314.21	108.642.30	12.7		12.7	169.941.41	West Virginia
Wisconsin	2.688.4	7.502.516.51	1.926.455.37	81	188.8	117.0	305.8	667.882.86	42.100.00	20.9	3.6	24.5	3.333.03	Wisconsin
Wyoming	2,084.0	3.046.599.22	1.386,017.49	79	239.2	134.7	373.9	450.311.63	5.449.71	39.3	74.1	113.4	4.974.85	Wyoming
Hawaii	78.4	2.013,891.20	1.108,383.10	33	55.4		55.4	508.751.81	314,307.62	10.7	1.9	12.6	748,813.81	Hawaii
TOTALS	103.873.6	250.723.571.82	101.098.486.94	68	9,152.4	3,825.6	12,978.0	51,976,408.39	15.613.731.70	2,249.1	978.4	3,227.5	25,583,269.10	TOTALS
Construction completed		157.635.000.00	69.160,000.00											
Balance uncompleted		93,089.000.00	31,938,000.00											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE 1932

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 11 ▽ JANUARY, 1933

SKYLINE DRIVE IN SHENANDOAH NATIONAL PARK

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 11 January, 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

SURFACE TREATMENT OF TOPSOIL ROADS[1]

Reported by **J. S. WILLIAMSON**, Assistant to the State Highway Engineer, South Carolina State Highway Department, and **PAUL F. CRITZ**, Associate Highway Engineer, United States Bureau of Public Roads

ABSTRACT.—This project, approximately 8 miles in length, was constructed in May and June, 1925, on United States Route 176, between the south limits of Inman and the North Carolina State line. The surface treatment consisted of a prime coat of one-quarter gallon per square yard, a second application of one-third gallon per square yard, and 45 pounds of cover material. Cut-back asphaltic oil and 8–13 viscosity (Engler) tar were used as primers. The materials used for second application were asphalt of 150–200 penetration and tar, both applied hot. One-fourth to one inch crushed granite was used as cover material on all sections except a part of section 9, on which ¼-inch chats were used, and parts of sections 9 and 10, on which ¼–1 inch slag was used.

Seal coats were applied to a portion of the experiment in November, 1925, and the remainder was sealed in March, 1926. The seal consisted of one-fifth gallon of bituminous material, and cover. The following materials were used on different sections and parts of sections: 18–25 viscosity tar, 8–13 viscosity tar, quick-drying asphaltic oil, and slow-drying asphaltic oil. Sand and ¼–¾ inch crushed granite were used as cover.

Portions of sections 7, 8, 10, and 11 required re-treatment in 1928, and part of section 11 in 1929. In 1930 portions of sections 8 and 11 were given treatments for the purpose of providing a nonskid surface. Maintenance was continued until July 1931, when the road was abandoned for a new location. The average costs for maintenance and re-treatments over the 6-year period amounted to $324 per mile, or $270 per mile exclusive of the cost of nonskid treatments applied to sections 8 and 11. The initial cost of surface treatment, including seal, varied from 17.67 to 24.43 cents per square yard.

Two materials failed to prove satisfactory, the cut-back asphaltic oil which was used as a primer and the slow-drying asphaltic oil used in the seal treatment. The former was a combination of a heavy asphaltic base and a highly volatile distillate. Because of its high original viscosity, increased by the loss of distillate on application, there was little penetration. The slow-drying oil developed a surface that was slippery in wet weather, and its use has been largely discontinued in favor of tars and quick-drying asphalts.

At the termination of the project all the sections were in good condition and indicated that they would have continued to give excellent service. Average traffic on this road was in the neighborhood of 600 vehicles per day in 1924 prior to treatment, and about 950 in 1931.

Beginning in the summer of 1923 the South Carolina State Highway Department undertook to develop an inexpensive bituminous treatment for use on the better class of topsoil and sand-clay roads, included in the State system. It was hoped to provide an all-weather surface which would eliminate the dust nuisance and carry comparatively dense motor traffic without excessive maintenance either for an indefinite period or, for certain roads, until such a time as funds might become available for providing a higher type of pavement.

A considerable mileage of topsoil and sand-clay roads had already been constructed which, under favorable conditions, were proving entirely adequate. However, on some roads, because of the amount of traffic or the character of the soil or both, it was impossible to provide a satisfactory riding surface free from objectionable dust in spite of excessive maintenance. It was to design a surface for such roads that a study of bituminous treatment was carried on by the State in 1923 and 1924.

The first series of experimental surfaces was constructed in 1923 and embraced six short sections. Five of these were constructed by the penetration method, the sixth by surface treatment. The conclusions drawn from this group of experiments were that the penetration type was less practical than the thin mat type of surface treatment, and that the degree of success obtained with the latter was dependent upon the quality of its supporting base.

The second series, built during 1924, covered six sections surface treated with heavy oils and tars as binding materials, with and without light oil and tar priming coats. The binding materials were applied at the rate of approximately a half gallon and the priming materials, when used, at about one-fourth gallon per square yard. Within a year after construction almost half of the areas on which the priming coat had been omitted required scarifying and re-treating, as did approximately one-third the area of the section on which a light, asphaltic oil was used as a primer. The two sections on which a light tar prime coat was used apparently remained in better condition as the percentage of failure recorded was much less.

In this early experimenting it was noted that some combinations of bituminous materials were satisfactory for some soil conditions but were not so satisfactory under other conditions. The form of treatment, however, gave such promising results in general that the highway department in cooperation with the Bureau of Public Roads constructed an experimental project to study the feasibility of surface treating topsoil or sand-clay roads using various types and grades of bituminous materials. The preliminary studies here briefly touched upon are described in detail elsewhere,[2] and are not included in this report, which covers only the cooperative experimental project referred to above.

The cooperative project was built in May and June, 1925, on State Road 19, now designated as United States Route 176, between the south city limits of Inman and the North Carolina State line. It was approximately 8 miles in length and was divided into five sections. Bituminous treatment was applied over a width of 19 feet, except for a distance of 3,800 feet through the town of Inman, where its width was 25 feet. A report describing the construction and early behavior of the project was published in PUBLIC ROADS, volume 8, No. 9, November, 1927. The project was maintained jointly by the State and the bureau until July, 1931, when the road was abandoned for a new location and the construction of a high-type pavement. A record was kept of the cost and character of maintenance and of the service behavior of each section. Table 1 gives the location, description, and maintenance cost of the various sections.

As previous experience had taught that the surface treatments remained more satisfactory on some types of soil than on others, a study of the topsoil and subgrade was made prior to applying the bituminous surfaces, in order to determine if possible the properties which a satisfactory soil base should possess. It was observed that on a major portion of each section the road surface was smooth and well bonded but on the remaining areas it was rough, pot-holed, or in a loose condition. On that portion of section 11 between stations 729+00 and 791+25 the surface sealed badly during treatment work. This condition was caused, it was believed, by the finely crushed granite which had been spread over the surface to a depth of about 2 inches but which had not worked into the surfacing material and as a result was only partially bonded.

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[1] Final report on a cooperative experiment in bituminous surface treatment by the South Carolina State Highway Department and the Bureau of Public Roads.

[2] Surface Treatment of Roads, by N. S. Anderson, Proceedings of the Fifth Annual Asphalt Paving Conference.

TABLE 1.—*Construction details and maintenance costs*

ORIGINAL CONSTRUCTION

Section	Station	Area	Surface treatment, May and June, 1925			Seal				Cost per square yard
			Prime coat (¼ gallon per square yard)	Second application (⅓ gallon per square yard)	Cover (45 pounds per square yard)	Date applied	Stations sealed	Bituminous material (⅕ gallon per square yard)	Cover	
		Sq. yds.								*Cents*
7 [1]	0+00 to 82+50	19,950	8-13 viscosity tar	Tar, hot	¼ in.-1 in. granite.	November, 1925. March, 1926.	38+00-82+50 0+00-38+00	18-25 viscosity tar 8-13 viscosity tar	Sand do	21.39
8	436 to 512	16,044	Cut-back asphaltic oil.	150-200 penetration asphalt, hot.	do	do	436-512	Quick-drying asphaltic oil.	do	17.67
9	512 to 517 517 to 533 533 to 536	1,056 3,378 633	do	Tar, hot	do ¼ in. chats ¼ in.-1 in. slag.	do	512-536	8-13 viscosity tar	do	24.43
10	536 to 553 553 to 622+50	3,589 14,672	8-13 viscosity tar	do	do ¼ in.-1 in. granite.	November, 1925. March, 1926.	574+50-622+50 536-553 553-574+50	18-25 viscosity tar 8-13 viscosity tar	do ¼ in.-¾ in. granite. Sand	22.84
11 [2]	622+50 to 791+25	35,414	do	150-200 penetration asphalt, hot.	do	November, 1925.	622+50-720 720-735 735-750 750-780 780-791+25	Slow-drying asphaltic oil.	do ¼ in.-¾ in. granite. Sand ¼ in.-¾ in. granite. Sand	18.04

MAINTENANCE AND RETREATMENT COSTS

[Cents per square yard]

Section	1925-26	1926-27	1927-28		1928-29		1929-30		1930-31		Total	Average annual	Annual surface cost per mile 20 feet wide
	Maintenance	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance			
7	3.08	2.11	[3] 0.99	1.30		1.22		1.57		1.33	11.60	1.93	$226
8	2.63	1.99	[3] 1.39	1.20		1.51	[3][4] 4.84	1.14	[3][4] 3.96	.74	19.40 [5] 10.60	3.23 [5] 1.77	379 [5] 208
9	2.37	.78		.62		.97		.73		1.42	6.89	1.15	135
10	4.00	2.07	[3] 3.07	.69		1.33		1.64		.66	13.46	2.24	263
11	5.56	3.22	[3] 1.31	2.01	[3] 2.41	1.41		1.36	[3][4] 5.86	.37	23.51 [5] 17.65	3.92 [5] 2.94	460 [5] 345
Average													324 [5] 270

[1] Treatment was made 25 feet wide from station 0+00 to 38+00; remainder of project was 19 feet wide.

[2] Station equation: Sta. 721+00=722+00.

[3] Section treated in part only but cost is proportioned over entire section.

[4] Treatment applied mainly to provide a nonskid surface.

[5] Costs exclusive of nonskid treatment.

TABLE 2.—*Analysis of topsoil and subgrade samples* [1]

SAMPLES TAKEN FROM THE TOPSOIL SURFACE

Laboratory No.	Section No.	Sample station	Mechanical analysis of granular material								Silt [2] 0.05 to 0.005 mm	Clay [2] less than 0.005 mm	Dye adsorption number	Moisture equivalent	Lower liquid limit	Plastic index	Shrinkage		Depth of topsoil surface at center line	Condition of the topsoil surface before treatment
			Retained ¼ inch	Passing ¼ inch, retained No. 10	Passing No. 10, retained No. 20	Passing No. 20, retained No. 60	Passing No. 60, retained No 100	Passing No 100, retained No. 200	Passing No. 200, retained No. 270	Total							Limit	Ratio		
1,562	7	5	4	7	8	26	9	12	4	70	17	13	7	8					7	Smooth, well bonded.
1,563	7	28	14	9	8	22	8	10	3	74	18	8	6	8					6	Loose, poorly bonded.
1,545	8	446	8	3	5	34	12	9	2	73	14	13	5	8	17	0	15	1.8	5	Rough, well bonded.
1,546	8	472	5	3	6	37	13	11	3	78	14	8	3	6	17	0	15	1.7	6	Loose, poorly bonded.
1,547	8	497	8	4	4	29	9	10	4	68	21	11	7	13	31	6	21	1.7	6	Good, well bonded.
1,549	9	525	11	5	5	38	14	11	0	84	12	4	2	4	15	0			6	Loose, poorly bonded.
1,551	10	552	3	4	6	33	10	11	4	71	15	14	9	10	23	0	19	1.7	5	Good, well bonded.
1,553	10	585	3	2	4	37	13	10	2	71	15	14	5	8	19	0	15	1.8	4	Good, fairly well bonded.
1,554	10	620	1	4	6	36	11	11	2	71	15	14	5	9	18	0	15	1.8	6	Do.
1,555	11	661	4	5	7	34	11	8	5	74	13	13	8	8			16	1.7	8	Good, well bonded.
1,556	11	693	8	10	7	30	11	9	2	77	15	8	4	7	18	0	14	1.7		Smooth, poorly bonded.
1,558	11	726	2	7	7	26	10	12	2	66	18	16	7	12	23	0	19	1.6	5	Fairly smooth, fairly well bonded.
1,559	11	757	8	9	6	25	12	11	5	76	20	4	5	7	17	0	17	1.7	5	Loose, very poorly bonded.
1,560	11	785	2	5	6	30	16	13	4	76	16	8	5	8	20	0	18	1.7		Do.

SAMPLES TAKEN FROM THE SUBGRADE

Laboratory No.	Section No.	Sample station	Retained ¼ inch	Passing ¼ inch, retained No. 10	Passing No. 10, retained No. 20	Passing No. 20, retained No. 60	Passing No. 60, retained No 100	Passing No 100, retained No. 200	Passing No. 200, retained No. 270	Total	Silt [2] 0.05 to 0.005 mm	Clay [2] less than 0.005 mm	Dye adsorption number	Moisture equivalent	Lower liquid limit	Plastic index	Shrinkage Limit	Shrinkage Ratio	Depth of topsoil surface at center line	Condition of the topsoil surface before treatment
1,565	7	49	0	1	1	26	9	8	4	49	16	35	10	21	44	22	27	1.5		
1,548	8	497	0	2	4	26	8	9	3	52	14	34	11	21	37	12	26	1.6		
1,552	10	552	5	4	6	29	10	12	4	70	17	13	7	.13	30	0	23	1.6		
1,557	11	693	1	5	5	23	7	8	4	53	19	28	8	14	33	15	21	1.7		

[1] According to the present laboratory method of analysis, the topsoil materials would most probably fall in the friable varieties of the A-2 soil group, while the subgrade materials would be classified as follows: No. 1,552, friable A-2; No. 1,557, plastic A-2; Nos. 1,548 and 1,565, A-7

[2] Although originally determined by methods not now in use, the values given herein are corrected to conform to present laboratory practice of defining silt and clay.

TABLE 3.—*Analyses of the tars used in the original construction*

Laboratory No.	27,153	27,154	27,198	27,199	27,201	27,706	27,769
Section	7	7	9 and 10	10 and 11	11	7 and 10	7,9 and 10
Location of use (stations)	0+00 to 82+50	0+00 to 82+50	512 to 536, 536 to 622 +50	536 to 630, 729 to 791 +25	630 to 729	38+00 to 82+50, 574 +00 to 622 +00	0+00 to 38+00, 512 to 574
Purpose	Prime coat	Second application	Second application	Prime coat	Prime coat	Seal coat	Seal coat
Specific gravity, 25°/25° C.	1.186	1.23	1.231	1.169	1.158	1.176	1.145
Specific viscosity, Engler, at 25° C.	---	---	---	48.7	36.7	---	53.8
Specific viscosity, Engler, at 40° C.	12.4	---	---	12	11.3	18.9	11.1
Float test at 32° C. seconds	---	167	182	---	---	---	---
Float test at 50° C. do	---	69	---	---	---	---	---
Bitumen soluble in CS_2 per cent	92.2	88.5	81.8	88	89.6	---	94.75
Free carbon do	7.7	11.4	18.1	11.8	10.3	8.8	5.19
Inorganic matter insoluble per cent	.1	.1	.1	.2	.1	.1	---
Distillation—							
Water do	1.2	0	0	2	2.4	1.36	2.57
Distilled to 170° C. do	2.29	0	0	2.79	4.21	---	---
170° to 235° C. do	7.07	2.25	2.47	9.35	6.25	4.83	8.80
235° to 270° C. do	10.83	7.10	6.54	10.90	9.98	12.51	12.76
270° to 300° C. do	6.49	3.66	4.73	5.10	5.83	6.24	6.76
Residue do	73.28	87.15	86.60	71.84	73.79	75.33	69.04
Softening point of residue, ° C.	34	42	49	40	38	30	38

TABLE 4.—*Analyses of the asphaltic materials used in the original construction*

Laboratory number	27,197	27,200	27,202	27,203	27,705	27,760
Section	8 and 9	8	11	11	11	8
Location of use (stations)	436 to 512, 512 to 536	436+00 to 512	700 to 791+25	622+50 to 700	622+50 to 791+25	436+00 to 512+00
Purpose	Prime coat	Second application	Second application	Second application	Seal coat	Seal coat
Specific gravity 25°/25° C.	0.950	1.031	1.032	1.031	0.9473	0.9439
Flash point ° C.	28	235	235	235	90	30
Burning point ° C.	31	---	---	---	150	55
Specific viscosity, Engler, at 25° C.	329	---	---	---	169.3	84.7
Specific viscosity, Engler, at 40° C.	117	---	---	---	41.19	31.8
Specific viscosity, Engler, at 100° C.	---	336	327	332	---	---
Penetration, 25° C., 100 grams, 5 seconds	---	165	161	164	---	---
Softening point ° C.	---	40	40	40	---	---
Loss, 163° C., 5 hours, 20 grams per cent	23	---	---	---	9.6	32.2
Residue, float test, 32° C. seconds	4,122	---	---	---	---	---
Residue, float test, 50° C. do	360	---	---	---	---	---
Loss, 163° C., 5 hours, 50 grams per cent	21.3	.17	.14	.16	6.1	27.4
Residue, float test, 32° C. seconds	852	---	---	---	---	---
Residue, float test, 50° C. do	237	---	---	---	---	---
Residue, penetration, 25° C., 100 grams, 5 sec.	---	139	147	146	---	---
Bitumen soluble in CS_2 per cent	99.8	99.8	99.8	99.8	99.9	100
Organic matter insoluble do	.2	.2	.2	.2	.1	---
Inorganic matter insoluble do	0	0	0	0	0	0
Bitumen insoluble in 86° Baumé naphtha per cent	20	23.6	24.2	23.5	13.3	17.6

The depth of the old topsoil wearing surface varied throughout the sections, ranging from 4 to 8 inches. Samples of the topsoil surfacing and of the subgrade were analyzed and the results are given in Table 2, together with a statement of the general appearance of the surface at the time the treatments were applied.

The bituminous materials were selected with a view to forming a stable mat of bitumen and stone which would adhere to the topsoil surface. In the early experiments it had been found that a bituminous material sufficiently viscous to hold a stone cover did not adhere satisfactorily to an untreated surface and that a priming application was highly desirable. The materials used on this project for priming were fluid products designed to penetrate well and to dry rapidly to permit early application of the second coat. The analyses of the various bituminous materials used are given in Tables 3 and 4.

The general procedure in constructing the bituminous surfaces was to sweep the topsoil surface clean of all loose and foreign material and then to apply about one-fourth gallon per square yard of the selected priming material. Holes or breaks occurring in the topsoil surface were repaired after priming, ordinarily by filling the depression with stone but in the case of very bad breaks a bituminous cold patch mixture was used. A considerable amount of such repair work was required on the north third, approximately, of section 11.

The prime coat was permitted to penetrate and set up for a day or two after which a more viscous bituminous material was applied at the rate of about one-third gallon per square yard. The surface was immediately covered by hand with about 45 pounds per square yard of the mineral cover selected and then rolled, after which the section was opened to traffic. During the first two or three weeks following construction it was necessary to respread the cover material thrown to the sides by traffic over those areas where bleeding and picking up indicated a deficiency of covering. Chats, which were waste concentrates from zinc mines and were mainly dolomite passing a ⅝-inch and retained on a No. 10 sieve, as well as slag and crushed granite, were used as cover materials on the location shown in Table 1.

During the fall and spring following construction all of the sections received a light re-treatment which served as a seal. The bituminous materials were of the same types as those used in the original construction but were less viscous. Sand was used as the cover material on all sections except a portion of section 10, on which crushed slag had been used originally, and two short lengths of section 11. On these portions three-fourths to one-fourth inch crushed granite was used. The nature and extent of these treatments as well as those applied subsequently to the various sections are shown in Figure 1.

HISTORY AND SERVICE BEHAVIOR OF THE SECTIONS

Section 7, stations 0+00 to 82+50. Construction: Prime coat, one-fourth gallon 8 to 13 viscosity[3] tar. Second application, one-third gallon hot tar. Cover, 45 pounds ¼ to 1 inch crushed granite. Seal application: Bituminous material, one-fifth gallon, 8 to 13 viscosity tar on stations 0 to 38, one-fifth gallon 18 to 25 viscosity tar on stations 38 to 82+50; cover, sand.

Cost, including seal coat, 21.39 cents per square yard.

The 3,800-foot portion of this section through Inman, because of its excellent condition in the fall following construction, was permitted to go through the first winter without a seal. On the remaining portion of the section, however, numerous surface breaks developed, and a seal was applied which consisted of one-fifth gallon of an 18 to 25 viscosity tar, applied

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[3] Unless otherwise noted all references to viscosity are expressed in terms of specific viscosity (Engler) at 40° C.

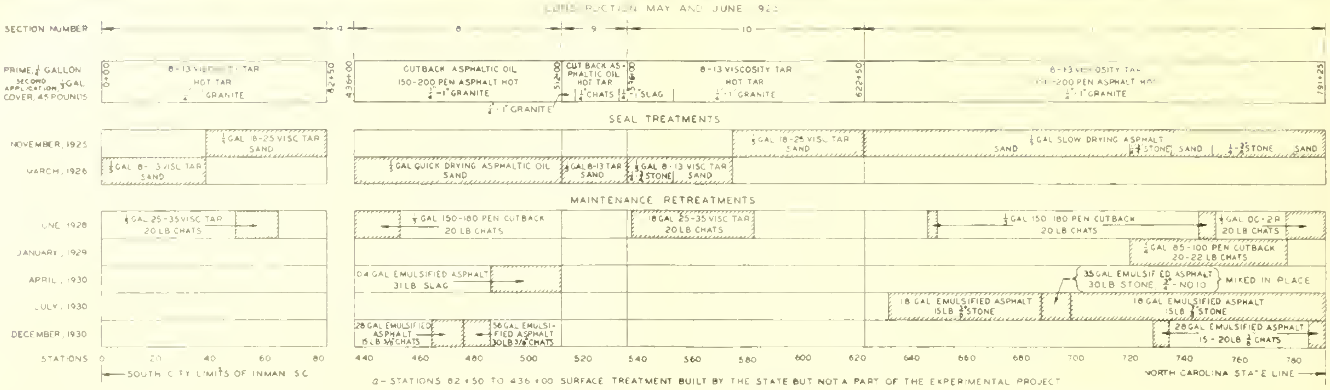

FIGURE 1.—CHART SHOWING NATURE AND EXTENT OF TREATMENTS APPLIED TO EXPERIMENTAL SECTIONS

cold, and a cover of sand. The area thus sealed remained in excellent condition during the winter but the portion which was not sealed in the fall required, by March, 1926, considerable patching; and it was then sealed with one-fifth gallon of an 8 to 13 viscosity tar and a cover of sand. The only re-treatment which this section received was applied in June, 1928, to that portion of the section between stations 49+25 and 64+50 and consisted of one-fifth gallon of 25 to 35 viscosity tar and 20 pounds of chats per square yard. Frequent inspections showed the section to have remained in excellent condition except for some roughness due partly to the original surface roughness of the base at the time of treatment and to a certain extent to the shoving resulting from the crushing of the fragile granite used. The section at the close of the experiment was probably the roughest of the group but a light drag or mixed-in-place re-treatment would have remedied this condition and, judging from its past history, would have created a satisfactory riding surface which could have been economically maintained indefinitely. The surface condition typical of the section is illustrated in Figure 2.

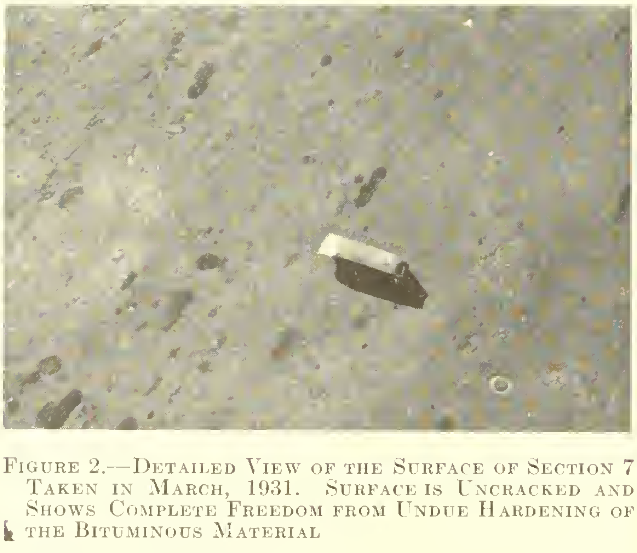

FIGURE 2.—DETAILED VIEW OF THE SURFACE OF SECTION 7 TAKEN IN MARCH, 1931. SURFACE IS UNCRACKED AND SHOWS COMPLETE FREEDOM FROM UNDUE HARDENING OF THE BITUMINOUS MATERIAL

Section 8, stations 436+00 to 512+00.—Construction: Prime coat, one-fourth gallon cut-back asphaltic oil. Second application, one-third gallon 150 to 200 penetration asphalt, hot. Cover, 45 pounds ¼ to 1 inch crushed granite. Seal application: Bituminous material, one-fifth gallon quick-drying asphaltic oil; cover, sand.

Cost, including seal coat, 17.67 cents per square yard.

This section was allowed to go through the winter following construction without a seal and as a result a large number of small patches were required during the winter and early spring. The entire section was sealed in March, 1926, with one-fifth gallon of quick-drying oil and a covering of sand. The only maintenance re-treatment required after that time was that given to the portion between stations 436 and 452+80. It was applied in June, 1928, and consisted of one-fifth gallon of cut-back asphalt and 20 pounds of chats per square yard. In 1930 the portion of the section between stations 464+50 and 512+00 was treated experimentally to eliminate slipperiness in wet weather and to provide a nonskid surface.

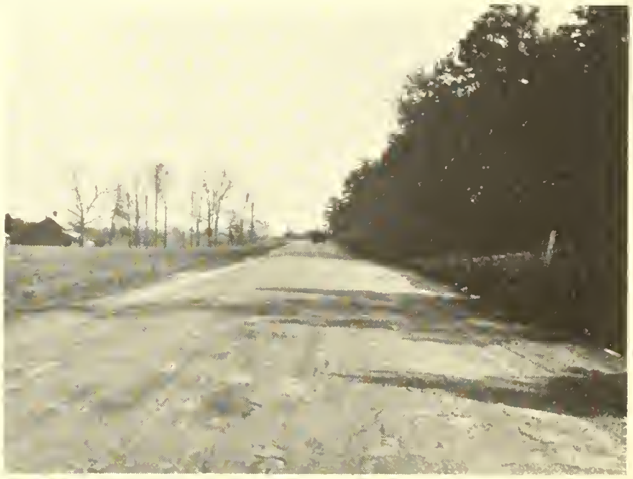

FIGURE 3.—FAILURE OF THE HEAVY NONSKID TREATMENT ON SECTION 8

Several methods of applying the treatment were tried out. One consisted of the application of a quick-breaking emulsion, spreading the cover material and rolling. This method was varied by applying a tack coat, then spreading and smoothing the aggregate and spreading another application of emulsion, depending upon the second application to penetrate through the cover stone to the tack coat. This treatment was generally not successful, because of the use of an amount of bituminous material insufficient to penetrate the stone cover. The result was that the surface peeled and raveled as shown in Figure 3. By another method the stone was spread, the emulsion applied, the two mixed, and the mixture spread and rolled. This method proved successful where sufficient bituminous material was used.

While a nonskid surface texture was obtained by each method, the first one seemed to serve the purpose most satisfactorily.

The portion of the section between stations 452+80 and 464+50 remained in excellent condition and received no treatment after March, 1926. Its condition in April, 1930, is illustrated in Figure 4.

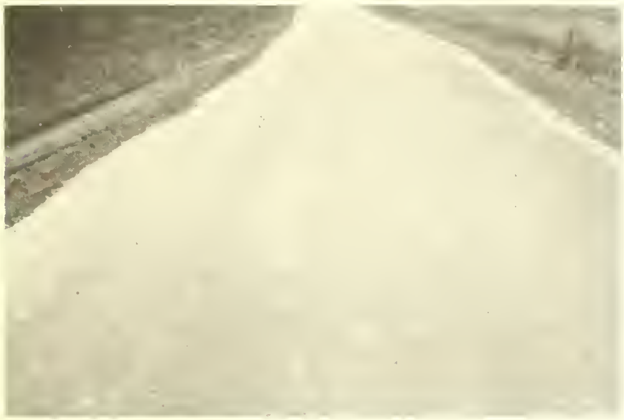

Figure 4. Portion of Section 8 Never Re-treated After Construction, Photographed in April, 1930

Except for the failure of the nonskid treatment mentioned the section as a whole remained in very good condition. Its cost was somewhat higher than that of section 9, primarily because of the necessity of providing the nonskid surface in 1930. The annual maintenance cost indicates that the section could have been continued in service at a reasonable cost.

Section 9, stations 512+00 to 536+00.—Construction: Prime coat, one-fourth gallon cut-back asphaltic oil. Second application, one-third gallon hot tar. Cover: 45 pounds ¼ to 1 inch crushed granite on stations 512 to 517; 45 pounds ¼ to 1 inch chats on stations 517 to 533; 45 pounds ¼ to 1 inch slag on stations 533 to 536. Seal application: Bituminous material, one-fifth gallon 8 to 13 viscosity tar; cover, sand.

Cost, including seal coat, 24.43 cents per square yard.

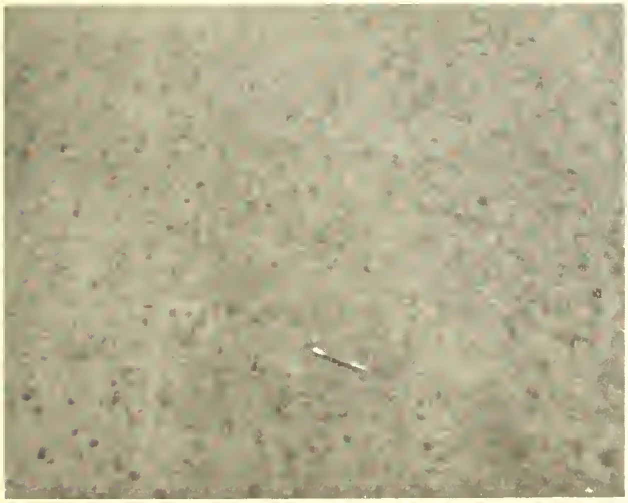

Figure 5.—Detailed View of Section 9, Which was Never Re-treated after Construction, Photographed in April, 1930

Section 9 also was permitted to go through the first winter without a seal as its surface was smooth and showed no defects. The portion on which chats were used resembled a Topeka pavement, while the slag-covered portion had a rough and granular texture. The former required some little patching during the winter but its condition in general was much better than the latter.

The entire section was given a seal coat in March, 1926, consisting of one-fifth gallon of 8 to 13 viscosity tar and a cover of sand. After that date the section received no further treatments. This fact, as well as the low maintenance cost, indicates its continued excellent behavior. It continued in the best condition of any of the sections, the chat-covered portion appearing somewhat better than the slag, which was more open. Figures 5 and 6 are typical illustrations of this section.

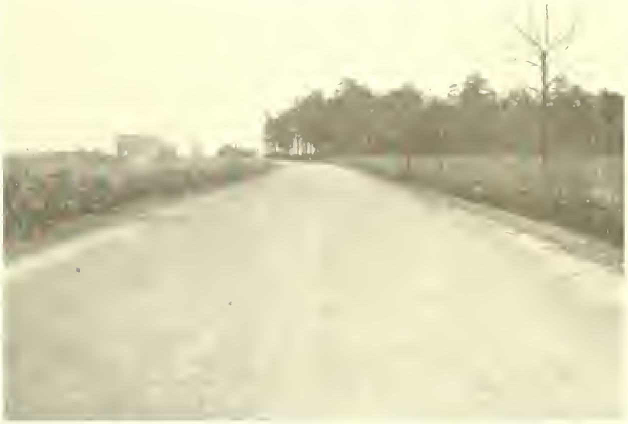

Figure 6.—General View of Section 9, Cover Material on Area Shown is Charts

Section 10, stations 536+00 to 622+50.—Construction: Prime coat, one-fourth gallon 8 to 13 viscosity tar. Second application, one-third gallon hot tar. Cover: 45 pounds ¼ to 1 inch slag on stations 536 to 553; 45 pounds ¼ to 1 inch crushed granite on stations 553 to 622+50. Seal application: Bituminous material, one-fifth gallon 8 to 13 viscosity tar on stations 536 to 574+50, one-fifth gallon 18 to 25 viscosity tar on stations 574+50 to 622+50; cover, ¼ to ¾ inch crushed granite on stations 536 to 553, sand on stations 553 to 622+50.

Cost, including seal, 22.84 cents per square yard.

The portion of this section between stations 574+50 and 622+50 was sealed in the fall after construction with an 18.9 viscosity tar and covered with sand. The remainder of the section was not sealed until the following spring as it was desired to retain the rough granular texture through the winter and also to obtain information as to the necessity for early sealing. During the winter many breaks occurred in the unsealed surface and these were repaired prior to applying the treatment in March, 1926. The treatment of this portion consisted of one-fifth gallon of 8 to 13 viscosity tar. Crushed granite ¼ to ¾ inch in size was used as the cover material on the portion between stations 536+00 and 553+00, originally covered with ¼ to 1 inch hard slag, while on the portion between stations 553+00 and 574+50, originally covered with crushed granite, sand was used.

After March, 1926, only one re-treatment was given and that was applied in June, 1928, to the portion of the section between stations 537+59 and 582+14, i. e., approximately the same portion as was sealed in March, 1926. The treatment consisted of an application of 0.18 gallon of 25 to 35 viscosity tar and a cover of 20 pounds of chats per square yard. The remainder of the section was not treated after the fall following its construction.

The section in general remained in excellent condition. The portion which was re-treated in 1928 had developed some cracks and showed indications of brittleness in the area where slag cover had originally been used. This may have been due to the fact that no allowance was made for the absorptive property of the slag and as a result the amount of second application material used developed a mat that was leaner than it should have been. Had the project been continued it is believed that a light re-treatment would have been desirable on this area. The portion never re-treated showed no need of immediate re-treatment. Figure 7 is a typical view showing its excellent condition.

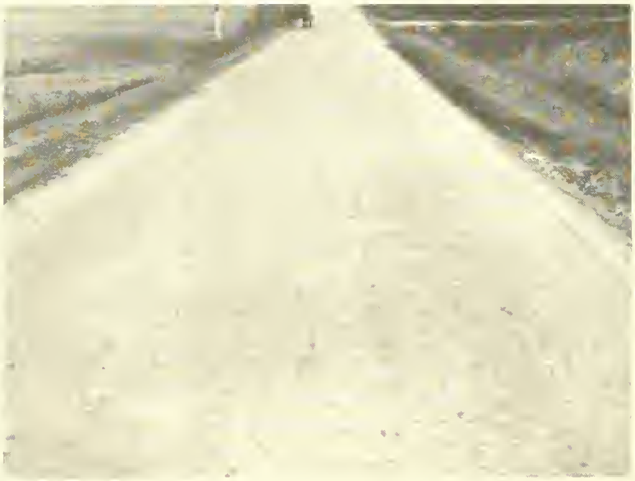

FIGURE 7.—TYPICAL CONDITION OF THE PORTION OF SECTION 10 WHICH WAS NEVER RE-TREATED

Section 11, stations[4] *622+50 to 791+25.*—Construction: Prime coat, one-fourth gallon 8 to 13 viscosity tar. Second application, one-third gallon 150 to 200 penetration asphalt, hot. Cover, 45 pounds one-fourth to 1 inch granite. Seal application: Bituminous material, one-fifth gallon slow-drying asphaltic oil; cover, sand on stations 622+50 to 720, 735 to 750, and 780 to 791+25, ¼ to ¾ inch crushed granite on stations 720 to 735 and 750 to 780.

Cost, including seal, 18.04 cents per square yard.

The north portion of the section from about station 720 to the end of the project at station 791+25, which lay on a steep grade, was covered with a 2-inch course of fine crushed granite prior to the surface treatment. It was expected that this material would stabilize the topsoil for later surface treatment and at the same time provide a more nonskid surface in wet weather. However, at the time of treatment, the primed surface was not well bonded, as indicated by the fact that it scaled and raveled under traffic. The prime was inadequate both as to the amount used and the method of application to overcome this condition and as a result considerable failure developed in the bituminous mat shortly after construction, necessitating a large amount of patching and numerous partial re-treatments.

The remainder of the section had a well-bonded topsoil surface comparable with that of the other sections and it remained in good condition, requiring no such extensive maintenance and re-treatments as did the north portion.

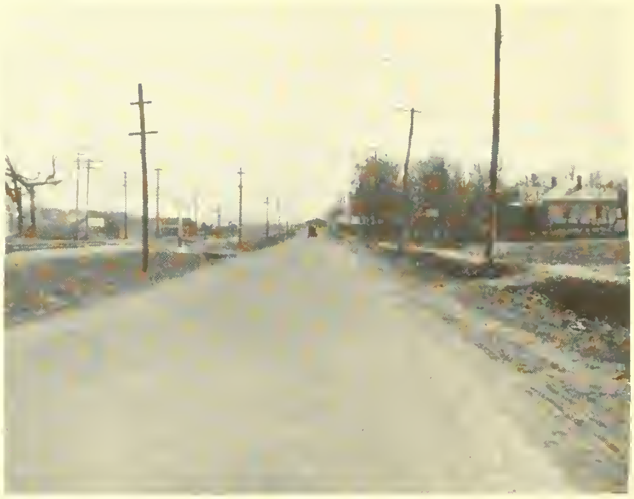

FIGURE 8.—VIEW OF THE SOUTH END OF SECTION 11, SHOWING THE EXCELLENT CONDITION OF THE PORTION NEVER RE-TREATED

In the November following construction the entire section was given a seal consisting of an application of one-fifth gallon of slow-drying asphaltic oil. Sand was used as cover material except for two short sections on which ¼ to ¾ inch granite was used. In June, 1928, re-treatments were applied to three short sections as follows: Stations 645+64 to 649+11, 744+71 to 751+05, and 776+75 to 791+25. The bituminous materials used were a 150 to 180 penetration asphalt cut-back and an asphaltic oil and were applied at the rate of one-fifth gallon per square yard. The cover material was chats spread at the rate of 20 pounds per square yard. In January, 1929, the area between stations 719+50 and 777+25 was re-treated using a quarter of a gallon of 85 to 100 penetration asphalt cut-back and 20 to 22 pounds of limestone chats. After this application all portions of the section were given one or two re-treatments, with the exception of the area between stations 622+50 and 631+25, which has never been re-treated. The re-treatments were applied primarily to develop a nonskid surface as the earlier seal treatment with road oil and sand cover had produced a surface that was very slippery in wet weather. An asphalt emulsion and ⅜-inch stone were applied by methods similar to those employed on section 8. These treatments were applied in July and December,

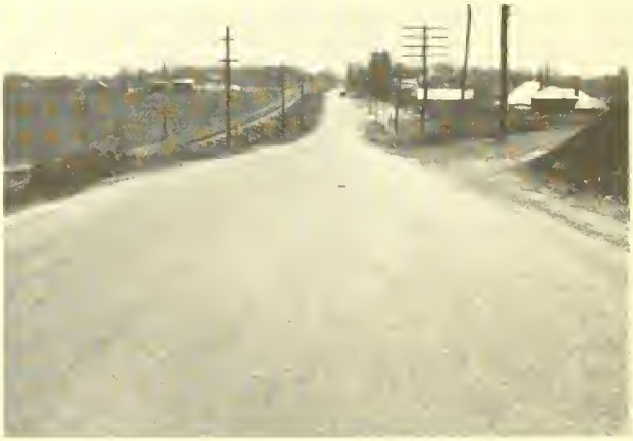

FIGURE 9.—TYPICAL APPEARANCE OF SECTION 11 AFTER APPLICATION OF THE NONSKID TREATMENT

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[4] Station equation, 721+00=722+00.

1930. The July treatment, except for the area between stations 687+25 and 698+00, upon which mixed-in-place seal coat was applied, was of the light seal type consisting of 0.18 gallon of emulsion and 15 pounds of ⅜-inch stone. The mixed-in-place seal was composed of 0.35 gallon of emulsion and 30 pounds of stone passing the ¾-inch and retained on the No. 10 sieve. Between stations 727+75 to 733+75 and 785+00 to 791+25 the surface was given an additional treatment in December of the same year. In this treatment 0.28 gallon of emulsion and 15 to 20 pounds of ⅜-inch chats were used. These treatments in general proved satisfactory, and the mixed-in-place seal produced considerable improvement in the surface smoothness.

Figure 10.—General View of North End of Project, Which was Located in Fairly Rough Country

The section as a whole was in better condition in July, 1931, than at any previous inspection. The portion which was never re-treated remained in excellent condition as illustrated in Figure 8. Figures 9 and 10 show other views of the section.

DISCUSSION

These experiments illustrate rather strikingly the fact that under average conditions surface treatments can be applied at moderate cost which will give excellent service over a period of years. They also offer an interesting comparison of methods of treatment in vogue six years ago with those of the present time. Although traffic had increased over 50 per cent during the period of the experiment its effect was not reflected in the cost of maintenance, which continued very uniform, as shown by the accumulated cost curves in Figure 11. At the termination of the project the sections were all in good condition and gave evidence that their record of excellent service would have continued.

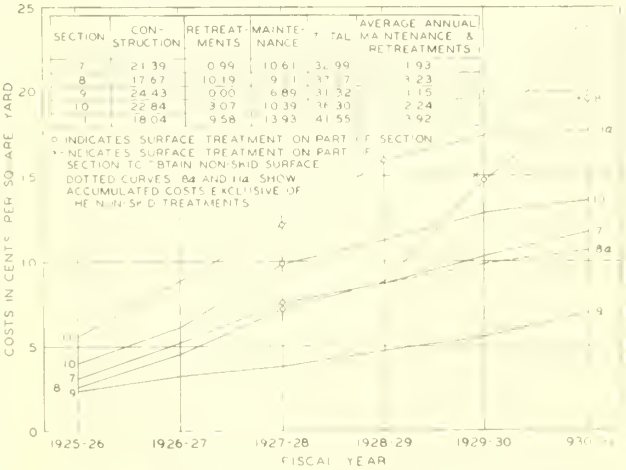
The subgrade and topsoil on the project were, in general, well suited to surface treatment as shown by their analyses, given in Table 2. One of the subgrades, sample No. 1552, section 10, was of the friable A-2 variety. No. 1557, from section 11, was of the plastic A-2 variety. The remaining two would be classed as the better variety of the A-7 group because of their higher liquid limits and lower sand contents. They are so close to the line separating the A-2 from the A-7 group that they also would be considered satisfactory subgrade materials under the favorable moisture conditions existing on this project. The topsoil surfacing, which averaged about 5 inches in depth, had the characteristics of the friable varieties of the A-2 group, material which provides an excellent base for surface treatment.

At the time the surface treatments were applied it was believed that a thoroughly bonded base was necessary and it was thought that the ideal base would be one which, untreated, would remain stable and would not ravel under traffic. This idea indirectly implied the use of a natural binder, such as clay, to maintain the bond, the amount varying with the characteristics of the binder and with the moisture conditions encountered.

It is known that capillary moisture stabilizes the cohesionless sands and other granular materials required for satisfactory road surfacing and bases for surface treatment. In the untreated road the surface portion is deficient in moisture because of evaporation, with the result that a clay binder is required to bind or cement the particles of granular material into a stable and wear-resisting surface. In the surface-treated road evaporation is largely prevented and the moisture content of the soil is increased by capillary action, with the result that the bond is sufficient to furnish the required stability not only in the top portion but throughout the depth of the base. Thus the need for clay binder is largely eliminated.

This does not mean that bond is not required in a base to be surface treated. As a matter of fact bond is required for stability in all types of soil roads whether they serve as bases or as surfaces. Thus the necessity for having a bonded surface upon which to apply the treatment is still recognized, but ideas have changed as to the method of obtaining this condition.

The binding properties of clay reach a maximum value at a certain moisture content and decrease with further increase in moisture. Consequently, the increase in moisture content due to surface treatment may not only reduce the binding property of the clay but may soften it to such an extent as to cause loss of stability in the base resulting in failure of the surface treatment. For this reason the quantity of clay required for an untreated clay-bound road, is often excessive when the road is to serve as a base for bituminous surface treatment.



SECTION	CON-STRUCTION	RETREAT-MENTS	MAINTE-NANCE	TOTAL	AVERAGE ANNUAL MAINTENANCE & RETREATMENTS
7	21.39	0.99	10.61	32.99	1.93
8	17.67	10.19	9.[illegible]	3[illegible]	3.23
9	24.43	0.00	6.89	31.32	1.15
10	22.84	3.07	10.39	36.30	2.24
11	18.04	9.58	13.93	41.55	3.92

○ INDICATES SURFACE TREATMENT ON PART OF SECTION
✳ INDICATES SURFACE TREATMENT ON PART OF SECTION TO OBTAIN NON-SKID SURFACE
DOTTED CURVES 8a AND 11a SHOW ACCUMULATED COSTS EXCLUSIVE OF THE NON-SKID TREATMENTS

Figure 11.—Accumulated Costs of Surface Treatments and Maintenance

The early unsatisfactory behavior of the north portion of section 11 could have been avoided by a more suitable priming treatment of the loosely bound top course. A greater amount of priming material with light surface mixing would have stabilized the base course to a greater depth than the light application of prime used and would have largely eliminated the

TABLE 5.—*Analyses of surface mats after six years' service*

Section No	10	11	11
Location sampled (station)	600	Center of road	625.
Original construction:			
Prime	¼ gallon 8–13 viscosity tar.	¼ gallon 8–13 viscosity tar.	¼ gallon 8–13 viscosity tar.
Second application	⅓ gallon hot tar.	⅓ gallon 150–200 penetration asphalt.	⅓ gallon 150–200 penetration asphalt.
Cover	¼–1 inch stone, 45 pounds.	¼–1 inch stone, 45 pounds.	¼–1 inch stone, 45 pounds.
Seal application—			
Bituminous material.	⅕ gallon 18–25 viscosity tar.	⅕ gallon slow-drying oil.	⅕ gallon slow-drying oil.
Cover	Sand	Sand	Sand.
Thickness of surface mat	⅜ inch	⅜ inch	⅜ inch.
Laboratory number of sample.	34370	34371	34372.

ANALYSIS

Section No	10	11	11
	Per cent	*Per cent*	*Per cent*
Bitumen extracted	4.5	5.9	8.4
Passing ¾ inch, retained ½ inch	8.5	10.5	3.5
Passing ½ inch, retained ¼ inch	16.9	30.2	26.4
Passing ¼ inch, retained No. 10 sieve	17.8	10.4	11.4
Passing No. 10 sieve, retained No. 20 sieve	9.9	5.3	7.0
Passing No. 20 sieve, retained No. 30 sieve	4.9	4.3	4.4
Passing No. 30 sieve, retained No. 40 sieve	7.1	5.3	6.4
Passing No. 40 sieve, retained No 50 sieve	5.9	4.5	5.4
Passing No. 50 sieve, retained No. 80 sieve	8.0	7.0	8.2
Passing No. 80 sieve, retained No. 100 sieve	2.8	2.9	3.2
Passing No. 100 sieve, retained No. 200 sieve	5.3	5.3	5.8
Passing No. 200 sieve	8.4	8.4	9.9
	100.0	100.0	100.0
Consistency of extracted bitumen:			
Float test at 32° C seconds	255		
Float test at 50° C do	50		
Penetration at 25° C		76	72
Softening point ° C		55.2	55

failures resulting from the displacement of the base materials.

The bituminous materials used in the original construction, analyses of which are given in Tables 3 and 4, were, with two exceptions, of the same types that are used to-day for similar work. The two exceptions are the quick-drying asphaltic material used as a prime and the slow-drying material used in the seal treatment.

The asphaltic primer was a combination of a heavy asphaltic base and a highly volatile distillate and had a specific viscosity at 40° C. of 117 which was approximately ten times that of the tar prime. Because of its high original viscosity, which was proportionately increased by the loss of distillate upon application, there was little penetration, and the heavy material remained on the surface as a mat. Had the prime been made up with a less volatile distillate and reduced to the consistency satisfactory for priming purposes, much better penetration would undoubtedly have resulted, without the deposition of heavy material on the surface.

The 8 to 13 viscosity tar used is still considered highly satisfactory as a priming material. Asphaltic primers also are used and the type favored is a material of low viscosity composed of a base and distillate such as to insure penetration and the development of a residue after penetration which will harden and develop some cementing value. The residue is not required, however, to serve as a binder to hold a cover of gravel or stone chips.

The other material which did not prove satisfactory was the slow-drying asphaltic oil used in the seal treatment on section 11. This material, according to its analysis, showed a low loss at 163° C. and an apparently soft residue. In combination with the sand cover it developed a surface that was slippery in wet weather. Because of this tendency its use has been largely discontinued as a seal material in favor of tars or quick drying asphalts.

There has been a tendency in recent years to use for re-treatment cold surface treatment material of viscosity higher than that of those formerly used. The greater viscosity produces stiffening qualities more nearly approaching those of materials applied hot, while at the same time permitting more manipulation during construction for the purpose of improving the riding qualities of the surface.

Cold application materials, compared with those applied hot, seem to insure in greater degree and for a longer period of time the maintenance of a nonskid surface. In this respect, however, it must be noted that the tar-treated sections in this experiment retained an excellent nonskid surface throughout the entire period, and did not require any treatment on this account.

To study the physical changes which might have occurred in the bituminous materials after six years of exposure to traffic and atmospheric conditions, samples of the surface mat above the primed base were taken from areas of sections 10 and 11 which had received no treatment subsequent to the seal following construction. A summary of the materials originally composing the samples and their analyses are given in Table 5.

With reference to the sample taken from section 10 which contained only tar, the percentage of bitumen extracted does not indicate the amount of the bituminous material contained in the sample, as the free carbon remained with the aggregate upon extraction. The original analyses of the tars used for the second application and seal coat show them to have had an average solubility in carbon disulphide of only about 85 per cent. Disregarding the increase in carbon content due to the loss of volatile matter and to weathering during the period of exposure, which would further decrease the percentage soluble in carbon disulphide, the actual amount of bituminous material present would be somewhat above 5.3 per cent, which agrees reasonably well with that contained in sample 34371 taken from section 11.

Sample 34372 was taken from an area on section 11 on which successive applications of bituminous material overlapped. In spite of its high bitumen content the area represented was in excellent condition and showed no evidence of rutting or shoving.

The tar extracted was composed of approximately five parts of a tar having an original float of 182 seconds at 32° C. and three parts of one having an original specific viscosity of 18.9 at 40° C. The resultant material after six years of service had, as noted, a float of 255 and 50 seconds at 32° C. and 50° C., respectively. Its final consistency therefore approximates that of a tar intermediate between materials suitable for hot surface treatment and bituminous macadam and would not indicate a tendency to harden unduly and become brittle although this tendency has been an assumed characteristic for this type of material. Apparently pronounced hardening is limited largely to the exposed surface and does not seem to affect greatly the major portion of the bituminous material in the mat even though the total thickness of the latter is relatively small.

The asphaltic material extracted was composed of approximately five parts of an asphalt cement having

(Continued on p. 184)

EFFECT OF SIZE OF SPECIMEN, SIZE OF AGGREGATE AND METHOD OF LOADING UPON THE UNIFORMITY OF FLEXURAL STRENGTH TESTS

Reported by W. F. KELLERMANN, Associate Materials Engineer, United States Bureau of Public Roads

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

A. Relative flexural strength.—1. The flexural strength of the specimens was influenced by the method of loading and computing the results. The three methods which were investigated gave values of modulus of rupture in the following descending order of magnitude:

(*a*) Center loading, bending moment computed at center of span;

(*b*) Center loading, bending moment computed at plane of fracture;

(*c*) Third-point loading.

2. The flexural strength of the concrete increased as the maximum size of the coarse aggregate became smaller.

3. With a constant ratio of span length to depth, higher flexural strengths were obtained on specimens having the smaller cross section.

4. With a constant cross section, higher flexural strengths were obtained on specimens having the shorter span.

B. Uniformity.—1. The uniformity of flexure test results was affected by the method of loading the specimen and by the method of computing the bending moment. The three methods used in these tests are given below in order of decreasing uniformity.

(*a*) Third-point loading;

(*b*) Center loading, bending moment computed at center of span;

(*c*) Center loading, bending moment computed at plane of fracture.

2. The uniformity of flexure test results was affected by the maximum size of the coarse aggregate employed, smaller aggregates producing somewhat more uniform results.

3. With a given ratio of span length to depth the specimens having the smaller cross section gave the more consistent results.

4. In tests of specimens of constant cross section tested on 18 and 27 inch spans, the shorter span produced the more uniform results in the case of the third-point loading and the center loading, with moment computed at center. In the case of the center loading with moment computed at plane of fracture, the reverse is generally true.

C. Recommendations.—It is recommended that the third-point method of loading and a cross section of 6 by 6 inches be standardized for laboratory work. In regard to span length no recommendation is made.

ALTHOUGH the compression test has long been used as a measure of the strength of Portland cement concrete and the procedure for making the test has been standardized, the results do not in all cases supply data for the correct design of a concrete pavement slab. Concrete for this purpose is usually investigated to determine its resistance to bending stresses and to do this it is necessary to test the concrete in flexure. While this test has been used regularly by a number of the State highway departments for several years it is only quite recently that there has been any standardized procedure for making it. As a result, it has been very difficult to correlate the large amount of test data which have been obtained by the States. The problem has been further complicated by the fact that, for purposes of control testing, it has become customary to make flexure tests directly on the job rather than in the laboratory. This has resulted in the development of a number of different field testing machines which differ considerably in design and, consequently, give different results. A report showing the extent to which variation in the type of testing machine may affect the results of flexure tests was published in Public Roads, volume 12, No. 12, February, 1932.

Variations in flexural strength may be due to many causes, ranging from differences in the quality of the concrete itself to differences in the procedure followed in making and testing the specimens and in computing the results. Variations due to methods of conducting the test may be caused by variations in any one or more of the following: (1) Method of loading; (2) rate of application of load; (3) cross-sectional dimension of specimen; (4) length of span; (5) method of computing bending moment.

Recently the committee on materials of the American Association of State Highway Officials, realizing the need for a standardized procedure, sponsored a series of cooperative tests to determine the variation in strength which different laboratories would obtain between their laboratory and field methods of testing. The results of these cooperative tests indicated clearly that it would be necessary to standardize the laboratory method before progress could be made with field standardization. A second series of cooperative tests were therefore conducted, the results of which were published in Public Roads, volume 12, No. 2, April, 1931. In this series all specimens were made and tested as simple beams in the laboratory, the load being applied at the third points by means of special apparatus designed for the purpose.

The need for a standard method of making and testing concrete beam specimens has been recognized by Committee C–9 of the American Society for Testing Materials, and as a result a tentative standard for this test was published in 1930 (A.S.T.M. designation C 78–30 T). This method proposes that the specimens be tested as simple beams with the load applied at the center of the span and may, therefore, give quite different results from the so-called third-point loading used by the American Association of State Highway Officials.

The tests herein reported were undertaken for the purpose of determining what effect this variation in method, as well as certain other variables, would have upon the results of flexure tests. The following variables were investigated: (*a*) Method of loading; (*b*) Size of specimen; (*c*) Span length; (*d*) Maximum size of coarse aggregate.

While the primary purpose of this investigation was to study the uniformity of the results, information was obtained showing how the above-mentioned variables affected the results quantitatively.

MATERIALS AND PROPORTIONS

Fine aggregate used in these tests was a bank-run sand having the physical properties shown in Table 1. This table also gives the results of the physical tests on the cement. The coarse aggregate was a limestone, which tends to produce concrete of relatively high flexural strength. It was screened into separate sizes and recombined into four definite gradings, as follows:

Percentage passing (square openings)

Grading	¼-inch	¾-inch	1¼-inch	2-inch	2½-inch
A	0	20	45	80	100
B	0	30	60	100	--------
C	0	40	100	--------	--------
D	0	100	--------	--------	--------

It will be observed from these grading combinations that there is a rather wide range in maximum size with a corresponding change in the void content of the material in a dry-rodded condition. Table 2 gives the results of physical tests on the coarse aggregate.

In designing the proportions for the various grading combinations the cement factor and consistency were held practically constant and the percentage of sand in the mix varied in order to compensate for a change in void content in the coarse aggregate. Data pertaining to the proportions and consistency will be found in Table 3.

FABRICATION AND STORAGE

All concrete was mixed in pans with shovels, one batch being of sufficient size to make one specimen. Three sizes of beam specimens were made—6 by 6 by 21 inches, 6 by 6 by 30 inches, and 8 by 8 by 27 inches. The 6 by 6 inch beams were molded in two layers, each layer rodded with a ⅝-inch rod, and spaded on the sides and ends. After the top layer had been poured the concrete was struck off and finished with a cork float. The 8 by 8 inch beams were molded in three layers. Each layer was rodded sixty times per square foot of area for all specimens. This required rodding the 6 by 6 by 21 inch beams fifty-four times per layer, the 6 by 6 by 30 inch beams seventy-five times per layer and the 8 by 8 by 27 inch beams ninety times per layer.

All specimens were covered with wet burlap immediately after fabrication and kept covered for approximately 24 hours, at which time they were removed from the molds and placed in a moist room for 27 days storage prior to test.

The procedure followed was to make 24 beams each working day. This constituted one round of tests and furnished one specimen for each combination of grading of aggregate, size of specimen, and method of test.

At the 28-day period, the specimens were tested in a 100,000-pound Universal testing machine as simple beams, the load being applied by a handwheel at a speed which would produce a stress of 150 pounds per square inch per minute in the extreme fibers (A. S. T. M. tentative standard).

Half of the beams were tested with the load applied at the third points and half with the load applied at the center of the span. Figure 1 shows the method of applying the load to the specimen while Figures 2 and 3 are photographs of a specimen set up in the testing machine. In computing the modulus of rupture in the case of specimens loaded at mid span, the bending moment was computed at the plane of fracture and also under the load at the center of the span. Two sets of results for modulus of rupture were therefore obtained on all specimens loaded at mid span. In addition to the variables in testing previously mentioned one set of 6 by 6 by 30 inch beams using ¾-inch aggregate was tested with the side instead of the bottom in tension as was the case with all other specimens.

TABLE 1.—*Physical properties of cement and fine aggregate*

CEMENT

Fineness, percentage retained on 200-mesh sieve	7. 8
Time of set (Gillmore):	
Initial	2 h. 52 m.
Final	4 h. 40 m.
Steam test for soundness	(1)
Normal consistency, per cent	24. 2
Tensile strength (1:3 Ottawa sand mortar):	
7 days ... pounds per square inch	380
28 days ... do	420

FINE AGGREGATE

Sieve analysis:	
Total retained on No. 4 sieve ... per cent	0
Total retained on No. 8 sieve ... do	11
Total retained on No. 16 sieve ... do	34
Total retained on No. 30 sieve ... do	61
Total retained on No. 50 sieve ... do	85
Total retained on No. 100 sieve ... do	94
Fineness modulus	2. 85
Silt and clay ... per cent	2. 8
Apparent specific gravity	2. 66
Weight in pounds per cubic foot (dry-rodded)	102
Organic matter (color test)	(1)
Strength ratio:	
7 days	132
28 days	113

Description: Sand consists essentially of angular grains of quartz, chert, gneiss, and sandstone.

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[1] Satisfactory.

TABLE 2.—*Physical properties of coarse aggregate*

Specific gravity	2. 72
Absorption ... per cent	. 12
Wear ... do	[1] . 44

Grading	Weight per cubic foot, dry-rodded	Voids
	Pounds	*Per cent*
A	103	39. 1
B	102	39. 7
C	101	40. 3
D	98	42

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[1] Not standard test; made with crushed rock.

TABLE 3.—*Proportions and consistency of concrete*

Grading	Maximum size aggregate	Proportions by dry-rodded volumes	W/C[1]	Cement factor	Mortar voids ratio	b/b_0
	Inches			*Bags per cubic yard*	*Per cent*	
A	2½	1:2. 0:3. 30	0. 75	6. 01	192	0. 735
B	2	1:2. 03:2. 27	. 76	6. 03	193	. 730
C	1¼	1:2. 09:3. 21	. 77	6. 04	198	. 718
D	¾	1:2. 21:3. 05	. 84	6. 04	211	. 682

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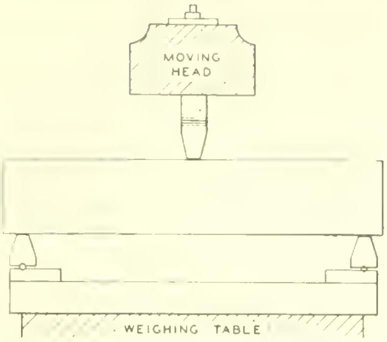
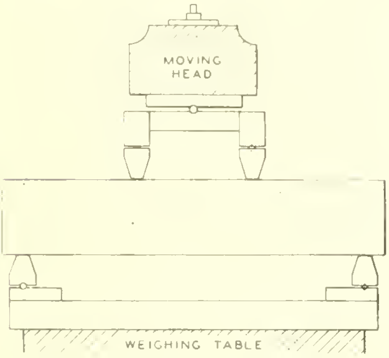
[1] W/C corrected for absorption. Consistency approximately 2½-inch slump.

ANALYSIS OF DATA

Detailed results of the strength tests, giving values for the individual specimens, were prepared in mimeograph form, but are not included in the report because of lack of space. They give the individual strength values for all 20 rounds of tests and the percentage variation of each value from the average. The average strengths and the average variations are also shown. While some values reported show a rather wide variation from the average for the group, they were not excluded because of the fact that it was the primary object of these tests to determine the extent of just such variations.



CENTER LOADING



THIRD POINT LOADING

Figure 1.—Apparatus for Testing Specimens in 100,000-Pound Testing Machine

Persons desiring to make a study of these detailed tables may obtain them upon request addressed to the bureau.

The average strengths and average deviations are also given in summary form in Table 4, and, for purposes of study, are shown in graphic form in Figures 4, 5, and 6.

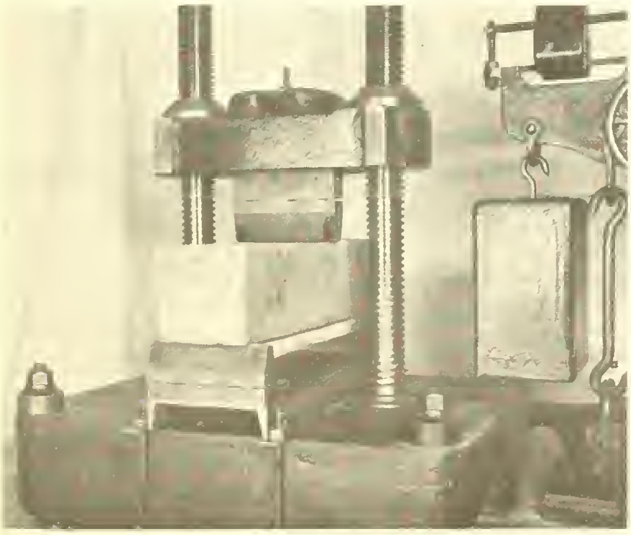


Figure 2.—Specimen Set Up in Testing Machine, Load Applied at Center of Span

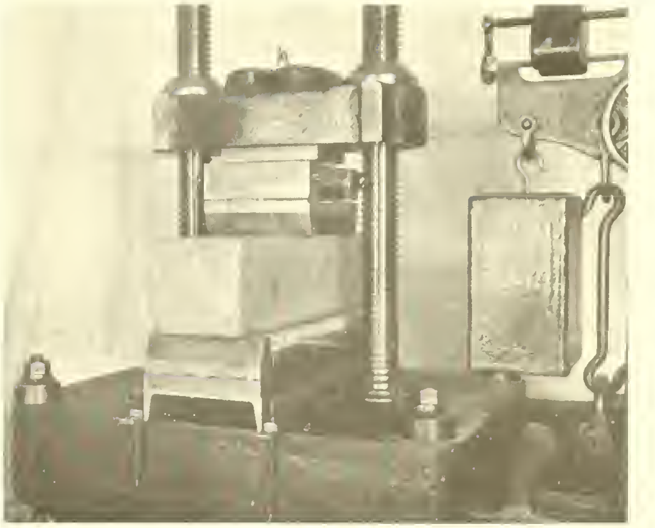


Figure 3.—Specimen Set Up in Testing Machine, Load Applied at Third Points

Table 4.—*Average values of flexural strength at 28 days and average percentage variations of individual values from the average*

Maximum size aggregate (inches)	Third-point loading		Center loading, moment at plane of fracture		Center loading, moment at center of span	
	Modulus of rupture	Variation	Modulus of rupture	Variation	Modulus of rupture	Variation
	Lbs. per sq. in.	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
6 BY 6 BY 21 INCH BEAMS—18-INCH SPAN						
2½	589	5.1	616	7.1	684	5.7
2	621	4.3	655	7.8	704	4.4
1¼	637	3.4	677	7.5	721	4.4
¾	672	4.1	705	6.0	753	3.6
Average		4.2		7.1		4.5
6 BY 6 BY 30 INCH BEAMS—27-INCH SPAN						
2½	572	5.6	608	7.9	646	7.4
2	573	7.3	632	6.2	663	5.0
1¼	605	5.7	646	6.3	684	5.1
¾	622	6.2	665	4.9	696	4.5
¾ [1]	641	5.2	652	6.3	693	4.2
Average		6.0		6.3		5.2
8 BY 8 BY 27 INCH BEAMS—24-INCH SPAN						
2½	534	5.2	573	10.3	620	8.5
2	562	4.7	590	7.0	646	5.5
1¼	576	4.2	628	6.8	669	5.3
Average		4.7		8.0		6.4
Grand average		5.1		7.0		5.3

[1] These specimens tested with side in tension, all others tested with bottom in tension.

The discussion deals with the effect which each of the variables studied has on (1) the relative average strength of the concrete and (2) the uniformity of strength within each group of individual specimens from which the average values were calculated. Figures 4, 5, and 6 have been plotted from the data shown in the tables in order to bring out certain relationship which appear to exist. These indications are discussed in the following order:

1. The effect of variations in the method of loading and in the method of computing the bending moment. (Fig. 4.)
2. The effect of changing maximum size of aggregate. (Fig. 4.)
3. The effect of changing the cross-sectional dimensions of the specimens when the ratio of depth of beam is to span constant at 1 to 3. (Fig. 5.)
4. The effect of changing the ratio of depth of beam to span length when the cross section is constant. (Fig. 6.)

The data plotted in the upper portion of each of the figures referred to above are the average strengths of the group of specimens representing each test condition. In most cases each point represents the average of 20 individual tests made on different days in accordance with the schedule which has been described. The data plotted in the lower part of each figure gives, for each test condition, the average per cent variation from the average of the group.

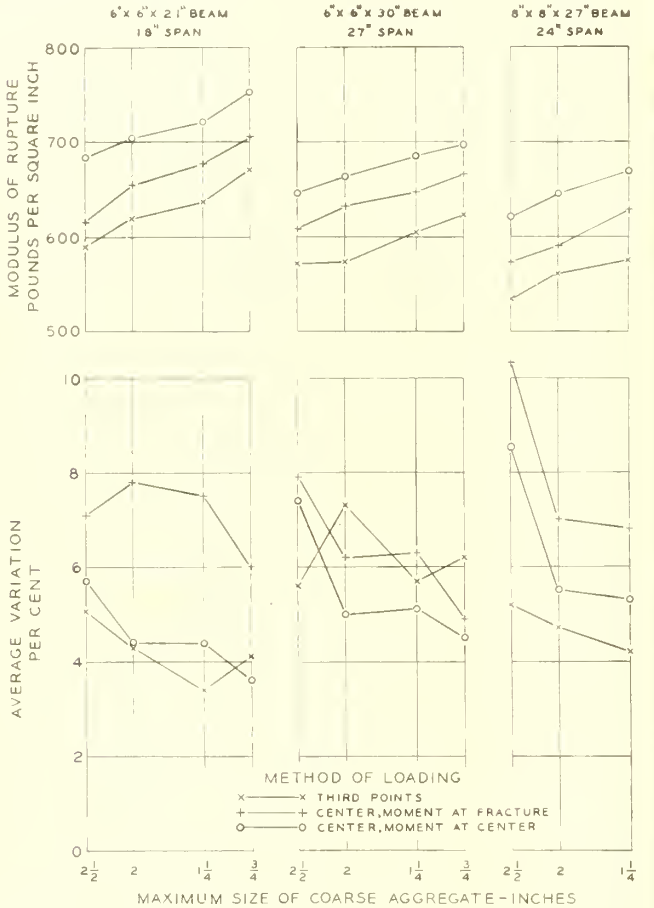


FIGURE 4.—EFFECT OF METHOD OF LOADING AND MAXIMUM SIZE OF COARSE AGGREGATE ON AVERAGE VALUES OF MODULUS OF RUPTURE AND AVERAGE PERCENTAGE VARIATION OF TEST RESULTS

Effect of method of loading and method of computing bending moment.—From Figure 4 it is evident that both the method of loading the specimen and the method of computing the bending moment have a marked effect upon the modulus of rupture. An examination of the curves in the upper portion of this chart shows that the method of loading the specimens at the third points produces the lowest results in flexure while the highest results are obtained by applying the load at the center of the span and computing the moment at the same place. Intermediate results are obtained by loading the specimen at the mid span and computing the moment at the plane of fracture. This relationship holds for all sizes of specimen and all maximum sizes of coarse aggregate without exception. If the specimens were fabricated from a material absolutely homogeneous in character and loaded at center of span, all of them would break at the center (plane of maximum moment). Concrete is no such material and very frequently when tested with the load applied at the center of the span the specimen will break at a plane somewhere between the point of application of the load and one of the reaction points. In such cases if the bending moment is computed at the plane of fracture, a lower flexural strength will be obtained than if the moment is computed at the center of span. For instance, in Figure 4, with 8 by 8 inch specimens, 2½-inch aggregate, the average value for the group was 573 pounds per square inch when the bending moment was computed at the plane of fracture, while the same specimens gave an average value of 620 pounds per square inch when the bending moment was computed at the center, a difference of 8 per cent. If individual cases are examined, greater differences will be noted. Thus, on round No. 1 of this series of tests a value of 508 pounds per square inch was obtained when the moment was computed at the plane of fracture while the corresponding value obtained when the moment was computed at the center of the span was 609 pounds per square inch, a difference of 20 per cent. This difference was caused by the specimen breaking 2 inches off center.

The uniformity of test results as affected by the method of loading is shown in the lower portion of Figure 4. It is interesting to note, in the two cases where the span length is three times the depth (left and right hand groups in the figure) that loading at the third points generally gives the most consistent results, with the center-loading method, moment computed at plane of fracture, always giving the least consistent results. For the span ratio of 1:4½ (6 by 6 by 30 inch beams) loading at center with moment computed at center gives in most cases the greatest uniformity.

Table 5, A shows the strengths obtained with center loading expressed as a percentage of the strength when tested at the third points. These percentages are shown for moments computed at plane of fracture and at the center of beam, as well as for each size of beam. It will be observed that the average strength of the concrete when tested with center loading exceeds that obtained from tests at the third points by about 7 per cent when the bending moment is computed at the plane of fracture; and by about 14 per cent when the moment is computed at the center of the beam.

Effect of maximum size of coarse aggregate.—These relationships are shown in Figure 4 for each size of beam and for each method of loading. In all cases, decreasing the maximum size of the aggregate increases the modulus of rupture. In this connection it is of interest to refer to Table 3 and to note that practically identical cement factors were used in the concretes containing the various maximum size aggregate and, moreover, that because of this fact the net water-cement ratio increased as the maximum size became smaller. This fact would lead one to assume that the actual strength would decrease instead of increase. However, the results are consistent and they would appear to indicate either that the relation between water-cement ratio and flexural strength varies with the maximum size, or that the cross sections of the test specimens were too small to indicate the true relation.

In considering the effect of size of aggregate upon the flexural strength it should be remembered that the proportions for the smaller aggregate contained a greater percentage of sand than those for the larger aggregate combinations. The percentage of sand in the mixes for gradings A, B, and C is in proportion to the voids in the coarse aggregate. Previous tests with an aggregate similar to the one used in this investigation where two gradings corresponding to gradings B and C were employed showed slightly higher strengths

Table 5.—*Strength ratios*

A.—EFFECT OF METHOD OF LOADING

Maximum size aggregate (inches)	Flexural strength expressed as percentage of that given by third-point loading					
	Center loading, moment at fracture			Center loading, moment at center		
	6 by 6 by 21 inch beams	6 by 6 by 30 inch beams	8 by 8 by 27 inch beams	6 by 6 by 21 inch beams	6 by 6 by 30 inch beams	8 by 8 by 27 inch beams
2½	105	106	107	116	113	116
2	105	110	105	113	116	115
1¼	106	107	109	113	113	116
¾	105	107		112	112	
Average	105	108	107	114	114	116

B.—EFFECT OF MAXIMUM SIZE OF AGGREGATE

Method of loading	Flexural strength expressed as percentage of strength of concrete containing 2½-inch maximum size aggregate		
	2-inch	1¼-inch	¾-inch
6 by 6 by 21 inch beams, 18-inch span:			
Third point	105	108	111
Center, moment at fracture	106	110	114
Center, moment at center	103	105	110
6 by 6 by 30 inch beams, 27-inch span:			
Third point	100	106	109
Center, moment at fracture	104	106	109
Center, moment at center	103	106	108
8 by 8 by 27 inch beams, 24-inch span:			
Third point	105	108	
Center, moment at fracture	103	110	
Center, moment at center	104	108	
Average	104	107	111

C. EFFECT OF SIZE OF BEAM

Maximum size aggregate (inches)	Flexural strength expressed as percentage of strength of 8 by 8 by 27 inch beams					
	6 by 6 by 21 inch beams			6 by 6 by 30 inch beams		
	Third-point loading	Center loading		Third-point loading	Center loading	
		Moment at fracture	Moment at center		Moment at fracture	Moment at center
2½	110	108	110	107	106	104
2	110	111	109	102	107	103
1¼	111	108	108	105	103	102
Average	110	109	109	105	105	103

D.—EFFECT OF LENGTH OF SPAN

Maximum size aggregate (inches)	Flexural strength of 6 by 6 by 21 inch beams, 18-inch span, expressed as percentage of strength of 6 by 6 by 30 inch beams, 27-inch span		
	Third-point loading	Center loading	
		Moment at fracture	Moment at center
2½	103	101	106
2	108	104	106
1¼	105	105	105
¾	108	108	108
Average	106	104	106

for the finer grading, even though there was a difference of 0.15 of a sack of cement per cubic yard of concrete in favor of the coarser grading. This present series indicates a possible change in flexural strength of as much as 14 per cent where the consistency and cement factor were held constant. (Table 5, B.)

As to uniformity, it will be observed that there is a distinct tendency for greater uniformity for the smaller maximum sizes. These relationships are shown in the lower part of Figure 4.

Table 5, B shows the effect of size of aggregate expressed as a percentage of the strength obtained with the 2½-inch material. It is noted that a decrease in maximum size to three-fourths inch increases the strength about 10 per cent; also that the effect of size is about the same for the two cross sections and for the two ratios of depth to span, except that in the latter case the shorter span shows a somewhat greater increase for the ¾-inch size. The method of loading, likewise, appears in general to exert little influence on this relation.

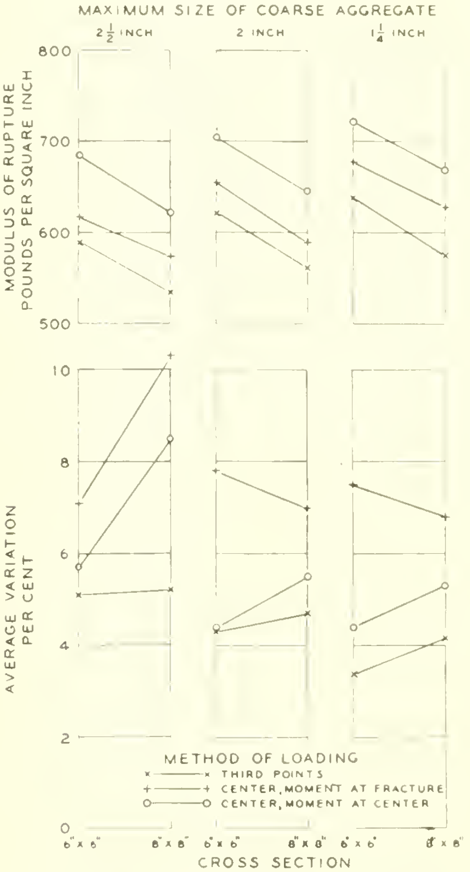


Figure 5.—Effect of Size of Beam on Average Values of Modulus of Rupture and Average Percentage Variation of Test Results. Span Is Three Times Depth

Effect of variations in cross section with ratio of depth to span constant.—The effect of increasing the cross-sectional dimensions of the specimen while maintaining the ratio of depth to span length constant at 1:3 is shown in Figure 5. It will be observed that for all three methods of loading, as well as for all maximum sizes of coarse aggregate, the smaller cross section gives the higher results. These observations are in line with the results of tests reported by a subcommittee of the committee on materials of the American Association of State Highway Officials, and published in the April, 1931, issue of Public Roads. In this series various depths and widths of beam specimens ranging from 4 by 4 inches to 10 by 10 inches were tested. It was observed that the modulus of rupture of the concrete decreased in direct proportion as the depth of the beam increased and it was pointed out that this dimension will have to be standardized if comparable results are to be expected.

From the standpoint of uniformity (lower portion of fig. 5) the results are not so consistent, although, in general, it may be said that the 6 by 6 inch beams give somewhat more uniform results than the 8 by 8 inch beams. This is true in all cases of third-point loading and also for center loading, moment at center.

The percentage differences are shown in Table 5, C, where the strength obtained on the 6 by 6 by 21 inch beams are shown as percentages of the strength of the 8 by 8 by 27 inch beams. The 6 by 6 inch cross section gave results about 10 per cent higher than the 8 by 8 inch section.

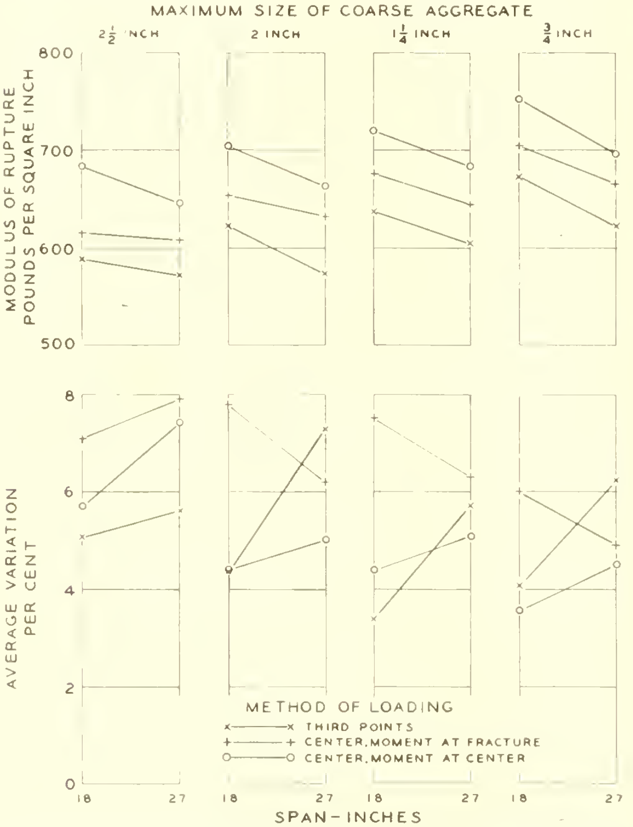


FIGURE 6.—EFFECT OF LENGTH OF SPAN ON AVERAGE VALUES OF MODULUS OF RUPTURE AND AVERAGE PERCENTAGE VARIATION OF TEST RESULTS, FOR 6 BY 6 INCH BEAMS

The strength obtained in tests of 6 by 6 by 30 inch beams are also shown in Table 5, C as percentages of the strengths of 8 by 8 by 27 inch beams. While the ratio of depth to span length is not the same in these two cases, it is of some significance that the 6 by 6 inch beams give somewhat higher strengths (approximately 5 per cent) than the 8 by 8 inch beams.

Effect of variations in ratio of depth of beam to span length for constant cross section.—This relationship, for each method of loading and for each maximum size of coarse aggregate, is shown in Figure 6. In all cases the specimens tested on the shorter span are somewhat higher in strength. In this respect the data do not check the results secured either by the committee on materials as reported in PUBLIC ROADS, volume 12, No. 2, April, 1931, or the work of the Portland Cement Association as reported by F. R. McMillan, director of research in 1928. McMillan, using beams 7 inches in depth by 10 inches in width, found the effect of span to be unimportant in the case of third-point loading. In the case of center loading, however, he found that the modulus of rupture decreased as the span increased.

When the data are analyzed with respect to uniformity, it will be observed that the shorter span length gives the most consistent results (lower average percentage variations) in all but three cases, all of which are for beams loaded at the center with moment calculated at plane of fracture. Each group of beams loaded at the third points show the most consistent results for the shorter span. This also holds for center loading with moment computed at the center of the beam.

Reference to Table 5, D will show that, if strengths are expressed on a percentage basis, the specimens tested on the shorter span (ratio of depth of beam to span length, 1:3) gave results about 5 per cent higher than those tested on the longer span (ratio 1:4½). The average percentage of increase is about the same for all methods of loading. For any given method of loading the percentage of increase, as affected by maximum size of aggregate, varies from 1 to 8 per cent, with a tendency for the concrete containing the 2½-inch aggregate to show somewhat lower ratios than the concrete containing the ¾-inch material. This will be discussed further in the next section.

DISCUSSION

The primary purpose of this series of tests was to determine which combination of the several variations in test procedure gave the most satisfactory results from the standpoint of uniformity. Reference to Table 4 will show that the grand average deviation for all specimens (236) tested at the third points was 5.1 per cent. The corresponding average for 239 specimens tested at the center of the span was 5.3 per cent when the moment was computed at the center but rose to 7 per cent for the same specimens when the moment was computed at the plane of fracture. Of the group tested at the third points, the 6 by 6 inch beams tested on 18-inch span gave the lowest average deviation, 4.2 per cent, as compared to 5.1 per cent for the entire group and 4.5 per cent for similar specimens tested with center loading, moment at center.

A possible reason for the higher variations obtained with the center-loading method where the moment was computed at the plane of fracture is the inability to determine just where this plane is with any degree of accuracy. With 2½-inch aggregate it is not hard to see that an error of one-half inch could easily be made in measuring the distance from the reaction to the plane of fracture. An illustration will show just what effect such an error would have upon the modulus of rupture. For instance, assume a beam specimen 8 by 8 inches tested on a 24-inch span, with the load applied at the center of the span. Assume also that the load required to break this specimen was 9,000 pounds and that the specimen broke 1 inch off center or 11 inches from one reaction.

Using the ordinary formula for computing the modulus of rupture and neglecting the dead weight of the beam itself, we obtain a value of 580 pounds per square inch when the moment arm is taken at 11 inches. An error in measurement of one-half inch, giving, say, a distance from the reaction of 10½ inches, would give a modulus of 554 pounds per square inch, a variation of 4.5 per cent due to this cause alone.

The simplest method is of course to assume the moment arm as one-half the span, regardless of where the beam breaks. This is the method ordinarily employed in practice. The value so computed is theoretically correct only for cases where failure occurs exactly at the center. For all other cases the true modulus of rupture at the section at which failure occurred will always be less than the apparent modulus computed at the center. For such cases the value reported as the modulus of rupture is not the stress which caused failure at all but simply the theoretical maximum fiber stress existing in the concrete at the center of the span at the moment of failure.

From the above discussion it would appear that both methods of computing the modulus of rupture where the specimen is loaded at the center are open to objection, the first because of the difficulty of accurately measuring the true moment arm, and the second because the value obtained does not represent the stress in the section of failure unless the specimen breaks exactly in the center, a condition not always observed.

By loading at the third points both of these objections are overcome. Provided the specimen breaks within the middle third of the span, it is not necessary to measure the moment arm because the moment is constant over the middle third and the value obtained will represent the true modulus of rupture of the section at failure.

In a further effort to throw light on the matter of uniformity the bar diagrams shown in Figure 7 have been prepared. These figures show the percentage of the total number of specimens in each group indicated which gave results varying more than 10 per cent from the average of the group. It is evident that, from this standpoint, the third-point method, using 6 by 6 inch beams tested on 18-inch span, gave the most consistent results. It should be remembered when studying these charts that each point represents a group of approximately 80 specimens in the case of the two 6 by 6 inch series and approximately 60 specimens in the case of the 8 by 8 inch series, and that the ¾-inch aggregate was not included in the fabrication of the 8 by 8 by 27 inch specimens.

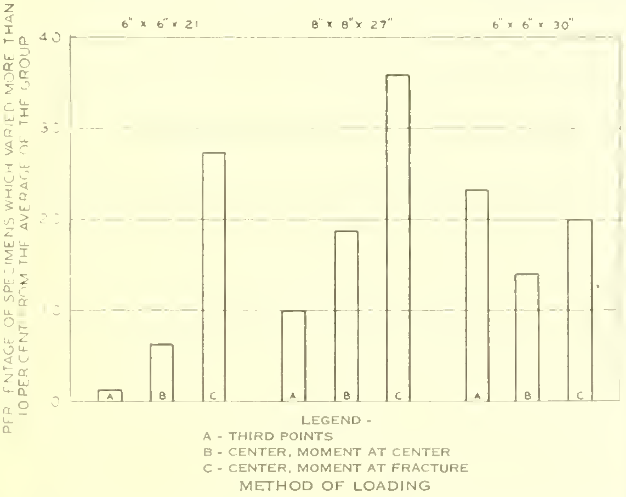


Figure 7.—Uniformity of Results for Different Test Conditions as Indicated by Percentage of Specimens Which Varied More than 10 Per Cent from Average of Group

The bar diagrams shown in Figure 8 afford a comparison between the results obtained from 6 by 6 by 30 inch specimens tested with the bottom (as molded) in tension, and those obtained from similar specimens tested with the side in tension. The differences in average modulus of rupture are slight. In the case of third-point loading the specimens tested with the side in tension gave somewhat the higher values. The reverse is true in the case of center loading, by both methods of computation.

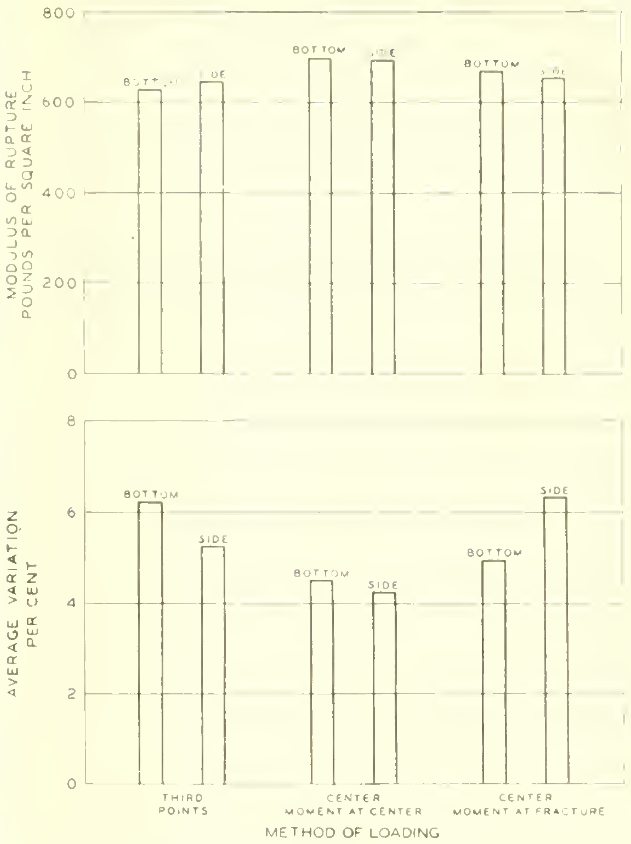


Figure 8.—Comparison of Results Given by Specimens Tested with Bottom, as Molded, in Tension, with Those Given by Specimens Tested with Side in Tension

The diagrams showing average variation indicate, in two cases out of three, a slight advantage in favor of the specimens tested with the side in tension. However, these tests are not conclusive enough to warrant a statement as to whether specimens should be tested with the side or the bottom in tension.

A short series of tests in which a rounded coarse aggregate was used in place of crushed stone was conducted for the purpose of giving some information regarding the effect of type of aggregate on strength and uniformity. Tests were made on one size of specimen only (6 by 6 by 30 inch) and on one maximum size of coarse aggregate (1¼-inch). Comparison of the method of loading at the third points and center loading shows the same trend as with the crushed aggregate. Loading at the third points gave the lowest strengths while center loading, moment taken at the center of the beam, gave the highest.

SUMMARY

From the data obtained in these tests the following indications have been summarized.

A. *Relative flexural strength.*—1. The flexural strength of the specimens was influenced by the method of loading and of computing the results. The three methods which were investigated gave values of the modulus of rupture in the following descending order of magnitude:

(*a*) Center loading, bending moment computed at center of span;

(*b*) Center loading, bending moment computed at plane of fracture;

(*c*) Third-point loading.

2. The flexural strength of the concrete increased as the maximum size of the coarse aggregate became smaller.

3. With a constant ratio of span length to depth, higher flexural strengths were obtained on specimens having the smaller cross section.

4. With a constant cross section, higher flexural strengths were obtained on specimens having the shorter span.

B. Uniformity.—1. The uniformity of flexure test results was affected by the method of loading the specimen and by the method of computing the bending moment. The three methods used in these tests are given below in order of decreasing uniformity.

(*a*) Third-point loading;

(*b*) Center loading, bending moment computed at center of span;

(*c*) Center loading, bending moment computed at plane of fracture.

2. The uniformity of flexure test results was affected by the maximum size of the coarse aggregate employed, smaller aggregates producing somewhat more uniform results.

3. With a given ratio of span length to depth the specimens having the smaller cross section gave the more consistent results.

4. In tests of specimens of constant cross section on 18 and 27 inch spans, the shorter span produced the more uniform results in the case of the third-point loading and the center loading with moment computed at center. In the case of the center loading with moment computed at plane of fracture, the reverse is generally true.

RECOMMENDATIONS

1. *Method of loading.*—From the standpoint of uniformity of test results there would appear to be but little choice between the third-point method of loading and a center loading with the moment computed at the center of the span. However, it is felt that the theoretical objections raised against the latter method, which have been discussed in this report, coupled with the fact that the uniformity of the results obtained by the third-point method of loading is as good as with the center loading, warrant a recommendation that the third-point method of loading be standardized for laboratory work.

2. *Cross section of specimen.*—The data indicate that specimens having cross-sectional dimensions of 6 by 6 inches give fully as uniform results as specimens of 8 by 8 inch cross section. Moreover, the decrease in uniformity due to increasing the maximum size of the coarse aggregate up to 2½ inches is so small as to be negligible from a practical point of view. It is recommended, therefore, that a cross section of 6 by 6 inches be standardized for laboratory testing.

3. *Span length.*—The results indicate a slight advantage from the standpoint of uniformity in favor of the shorter span. The data also indicate that the modulus of rupture is decreased by increasing the span. As far as the third-point method of loading is concerned, these results are in conflict with the results reported by both the committee on materials, American Association of State Highway Officials and the Portland Cement Association. In view of this fact, and also because only two span lengths were investigated, no recommendations regarding this dimension are made.

(Continued from p. 176)

originally a penetration of 164 and a softening point of 40° C., and three parts of an asphaltic oil having an original specific viscosity of 169 at 40° C. and 44 at 50° C. The resultant residue had a penetration of 76 and a softening point of 55° C., approximating a normal asphalt cement, and was fairly soft and plastic, indicating its ability to continue to hold the cover stone without excessive cracking in cold weather or bleeding in hot weather.

The cost of maintaining the wearing surface of the project through the 6-year period, while varying among the sections, is considerably less than that of an untreated topsoil carrying much less traffic. Data obtained in a survey of low-cost roads by C. N. Connor[5] indicate that the corresponding maintenance cost of untreated topsoil and sand-clay roads averages between $300 and $600 per mile annually and that such roads, ordinarily, serve economically and satisfactorily a maximum of about 400 vehicles daily, while under heavier traffic the maintenance cost increases rapidly.

Excluding the cost of the nonskid treatments which, while they obviously added somewhat to the betterment of the areas affected, were applied principally as a safety factor, the maintenance ranged from $135 to $345 per mile annually and averaged $270. Including the nonskid treatment the average annual cost was $324 per mile. While this cost may be higher than present-day maintenance of similar type, it should be recalled that little preliminary work was done on the road prior to applying the surface treatments and also that the project was made up of relatively short experimental sections. The crown, while satisfactory for the old sand-clay type, was excessive for a surface-treated road and is believed to have added somewhat to the cost of maintenance.

Traffic records show that in 1924 the road was carrying an average of 636 and a maximum of 800 vehicles daily based on a count through the summer months only. For the fiscal year 1930–31 a monthly count showed the sections to be carrying an average of 956 vehicles daily with a maximum of 1,402 which is an increase of over 50 per cent. As shown in Table 1 the maintenance cost of the sections, with the exception of the first year after construction which usually is relatively high, continued very uniform in spite of the great increase in traffic.

In addition to the decided economy of maintenance and the providing of a surface of the required carrying capacity, other advantages which can not be estimated in dollars and cents, such as reduced operating expense, elimination of dust, all-weather surfaces, and increased surface smoothness were obtained and could be retained indefinitely at a reasonable cost.

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[5] Pt. 2 of the Proceedings of the Seventh Annual Meeting of the Highway Research Board.

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No. 136D . .	Highway Bonds. 20 cents.
No. 347D . .	Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.
No. 532D . .	The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.
No. 583D . .	Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.
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MISCELLANEOUS CIRCULARS

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No. 109MC . .	Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters. 10 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . .	The results of Physical Tests of Road-Building Rock. 25 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They can not be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . .	Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927.)

Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

Report of a survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States. (1930.)

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, was printed in PUBLIC ROADS, vol. 13, No. 3, May, 1932. Copies of this list may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

DECEMBER 31,1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
					Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2.356.8	$ 4,556.923.95	$ 2.278.461.89	22	149.5	71.0	220.5	$ 55.684.41	$ 27.542.20	0.2		0.2	$ 3.780,068.45	Alabama
Arizona	1.199.2	2.949.843.07	1,413,090.53	68	89.6	167.9	257.5	87,905.53	17.581.10	15.6	1.2	16.8	204,794.94	Arizona
Arkansas	1,951.4	2,813.712.52	1.309,563.67	76	82.2	38.4	120.6	1.051,861.97	525.899.92	24.9		24.9	1.789,683.07	Arkansas
California	2,476.4	8.388.931.59	2.647.321.90	64	150.1	33.2	183.3	1.151,372.82	110.397.61	34.0		34.0	8.200.71	California
Colorado	1.726.3	2.895.098.75	1.383.452.79	77	120.8	51.0	171.8	1.030.442.59	463,699.06	53.4		53.4	829,661.17	Colorado
Connecticut	289.8	4.202.606.92	1.600.288.57	71	36.8		36.8	293.364.98	136,961.58	6.9		6.9	235.454.15	Connecticut
Delaware	374.5	591.801.95	148.953.90	74	16.2	11.2	27.4	217.621.10	40.500.00	10.9	6.0	16.9	43,564.26	Delaware
Florida	649.0	5.975.361.00	2.849.920.47	49	140.7	11.2	151.9	259.631.22	129.515.59	17.1		17.1	1.269.128.83	Florida
Georgia	3.184.0	5.667,220.89	2,292.790.32	50	163.6	161.9	325.5	605,806.82	259,140.89	23.3	38.7	62.0	136.800.60	Georgia
Idaho	1.483.7	2.659.799.54	1.135.270.07	81	136.1	156.3	292.4	552.631.35	176,247.73	14.1	19.3	33.4	88,957.93	Idaho
Illinois	3.049.2	20.367.431.33	8.175.776.16	80	630.5	33.0	663.5	2.339.991.15	672,202.48	88.2		88.2		Illinois
Indiana	1.992.4	8.646.426.35	4.023.788.55	91	311.7	3.0	314.7	1.123.467.35	111.500.00	59.2		59.2	215,980.36	Indiana
Iowa	3.501.8	5.628.149.98	1.066,185.38	92	278.4	40.0	318.4	359.559.51	23,060.00	25.0	1.2	26.2	11.498.31	Iowa
Kansas	3.826.8	3.878.723.94	1.792.236.37	61	265.2	68.6	333.8	1.777.322.69	146,560.84	111.1	42.9	154.0	209.093.30	Kansas
Kentucky	1,865.3	4.923.364.48	1.769.101.76	54	221.0	128.0	349.0	297,079.27	79,300.00	10.4		10.4	147.325.49	Kentucky
Louisiana	1.617.8	6.155,353.72	2,766.064.79	52	58.4	14.1	72.5	1.119.578.58	421.437.65	8.7		8.7	23.644.29	Louisiana
Maine	819.1	1,631,574.45	450.859.77	71	52.7	.3	53.0	824.783.83	85.446.97	17.8	1.9	19.7	67.851.99	Maine
Maryland	823.0	1.154,092.52	458,441.25	86	47.8	5.6	53.4	667.376.80	30.160.00	24.4		24.4	34.050.11	Maryland
Massachusetts	862.5	4.625.191.90	1.486.801.48	71	51.9	4.9	56.8	809.885.04	90.000.00	17.0		17.0	45.594.16	Massachusetts
Michigan	2.273.9	7.418,458.62	3.064.832.45	69	294.2	101.2	395.4	997.000.00	364.117.50	77.6		77.6	232.367.49	Michigan
Minnesota	4,338.0	4.636.721.39	131,628.91	84	169.1	120.0	289.1	114.649.27	100.00	1.3	.4	1.7	27.912.79	Minnesota
Mississippi	1.832.8	5.240,861.56	2.580.805.16	63	195.5	70.1	265.6	647.288.94	323,644.43	26.6	4.8	31.4	3.836.022.85	Mississippi
Missouri	3.177.6	2.892,126.89	694.216.47	72	111.6	14.3	125.9	613,660.53	166.419.71	43.7		43.7	389.002.63	Missouri
Montana	2.657.2	7.698.670.86	4.321.375.04	57	609.8	253.2	863.0	1,117,847.85	628.152.16	78.8	37.1	115.9	85.122.27	Montana
Nebraska	4.265.6	5.642.108.55	2.647.662.53	39	160.7	130.5	291.2	16.436.22	8.218.11	.5		.5	307.133.27	Nebraska
Nevada	1.333.9	1.683.271.03	925.075.93	85	29.6	128.0	157.6	236.318.92	26.384.56	9.1	20.3	29.4	69.842.56	Nevada
New Hampshire	427.1	950.750.66	416,351.61	77	27.8	3.5	31.3						170.907.95	New Hampshire
New Jersey	624.2	7.724.869.29	2.708.060.69	64	68.8	.5	69.3	79.106.70	19.718.17	2.7		2.7	6.811.52	New Jersey
New Mexico	2.229.0	2.782.515.32	1,289,139.72	52	164.7	78.6	243.3	403,963.67	177.474.14	4.4	26.8	31.2	305.581.88	New Mexico
New York	3.448.9	18,886.994.40	5.883.985.00	55	486.5	31.2	517.7	2.447.343.00	1.061.798.55	47.7	3.6	51.3	43,750.00	New York
North Carolina	2.259.2	2.693.213.57	1.348.145.58	40	312.9	16.9	329.8	1.752.055.40	877,865.13	216.9	.4	217.3	1,859.234.71	North Carolina
North Dakota	5.138.8	4.781,076.77	2.080.296.59	51	399.2	481.0	880.2	916,881.06	295.644.97	64.4	254.0	318.4	124.145.97	North Dakota
Ohio	2.991.8	7.413.241.34	2.227.149,23	82	191.7	74.2	265.9	3.597.828.00	734.522.60	43.4	6.4	49.8	42.544.41	Ohio
Oklahoma	2.383.8	4,106,670.40	2.042.943.66	77	203.9	48.4	252.3	2,109.809.89	440.091.91	114.8	30.0	144.8	562.753.00	Oklahoma
Oregon	1.602.5	3.541.954.56	1.503.216.02	56	102.7	84.3	187.0	922,864.43	303.995.15	25.1	22.4	47.5	127,260.94	Oregon
Pennsylvania	3,135.2	10.120,469.97	3.406.989.18	65	379.4	9.8	389.2	3.634.638.17	847.799.27	118.3	4.2	122.5	1.675.85	Pennsylvania
Rhode Island	263.8	1.226.843.60	432.058.05	54	25.0	4.3	29.3	285,171.25	103.490.86	5.6	.2	5.8		Rhode Island
South Carolina	1.996.5	2.968.789.31	1,142.092.73	54	160.1	119.2	279.3	609.448.09	192.953.34	12.4	31.5	43.9	2.200.00	South Carolina
South Dakota	4.076.2	4.353.209.43	1,975.343.14	76	330.0	280.9	610.9	68.178.30	30.549.67	14.6		14.6	87,786.58	South Dakota
Tennessee	1.690.3	4.184.392.57	2,091,178.22	33	132.5	49.9	182.4	1.234.074.82	617.037.39	40.8	14.8	55.6	705,399.78	Tennessee
Texas	7.782.6	17.146.700.12	7.214.864.71	62	724.0	463.0	1,187.0	6.081.700.24	1.697.301.02	297.4	143.4	440.8		Texas
Utah	1.205.4	1.927,206.25	918.241.76	78	115.0	89.2	204.2	360.887.34	108.477.74	34.5	20.3	54.8	247.576.96	Utah
Vermont	364.7	994,586.22	328.925.86	89	45.1		45.1	56.067.00	6.000.00	2.4		2.4	15.804.25	Vermont
Virginia	1.941.5	3.943.384.27	1.730,745.60	50	196.4	38.1	234.5	832.320.68	227,710.29	22.5	13.9	36.4	757.623.59	Virginia
Washington	1,318.8	2.306.174.10	812.112.10	52	95.3	1.1	96.4	567.014.18	174.000.00	20.3		20.3	197.723.59	Washington
West Virginia	913.0	3.375.497.98	1.307.832.99	61	116.8	8.5	125.3	274.081.75	64.622.41	8.3		8.3	188.766.75	West Virginia
Wisconsin	2.714.1	7.335.076.72	1.605.535.94	81	184.8	112.5	297.3	52.061.50	21.500.00		.9	.9	2,443.67	Wisconsin
Wyoming	2.048.1	3.114.484.77	1.199.108.82	79	256.6	183.7	440.3	115.762.14	2.104.28	9.1	21.8	30.9	5.328.32	Wyoming
Hawaii	78.4	2.176.281.06	1.179.014.06	31	60.1		60.1	314.782.03	243.992.92	5.9	1.9	7.8	749.436.92	Hawaii
TOTALS	104.561.9	250.978.210.43	98.257.297.37	63	9.353.0	3.995.7	13.348.7	45.084.608.68	13.314.239.90	1.911.3	770.3	2.681.6	20.293.512.62	TOTALS
Construction completed		145.427.000	62.296.000											
Balance uncompleted		105.551.000	35.961.000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

A 22,6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 13, NO. 12 FEBRUARY, 1933

AIR VIEW OF WOODWARD AVENUE, WAYNE COUNTY, MICHIGAN

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - See Page 2 of cover for prices

PUBLIC ROADS

▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 13, No. 12 February, 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

A SURVEY OF HIGHWAY TRANSPORTATION IN MICHIGAN

A REPORT OF HIGHWAY USAGE UPON TRUNK LINE, COUNTY AND TOWNSHIP HIGHWAYS AND UPON CITY STREETS OF MICHIGAN DURING 1930 AND 1931

By the Bureau of Public Roads, United States Department of Agriculture and the Michigan Highway Department

SUMMARY

Of the 85,080 miles of rural highways in Michigan, the trunk-line roads include 7,691 miles (exclusive of sections within incorporated areas), the county roads 17,175 miles, and the township roads 60,214 miles. The average traffic, in vehicle-miles per day, on these various systems, is as follows: Trunk-line roads, 8,804,656; county roads, 3,264,107; township roads, 1,327,801; all rural roads, 13,396,564. The average density of traffic on the trunk-line roads was found to be 1,144 vehicles per day; on the county roads 190 vehicles per day; and on the township roads 22 vehicles per day. About 14 per cent of the township mileage carries an average traffic of 5 vehicles per day or less, and 25 per cent carries 10 vehicles or less.

Of the traffic on township roads, 60 per cent originates within the local township and only 18 per cent originates outside the county. Of the traffic on county roads, 70 per cent originates inside the county and 30 per cent outside. Sixty per cent of the traffic on the trunk-line roads comes from outside the local county.

Approximately one-half of the traffic in the State, about 13,000,000 vehicle-miles per day, is on city streets. Of this total 69 per cent is local and 31 per cent nonlocal.

City streets are used by vehicles of rural origin to about the same extent as the township and county roads are used by vehicles of urban origin. The use of the trunk-line roads by urban and rural residents is approximately in proportion to the ratio of urban to rural population.

Expenditures on the various classes of highway in 1930-1931, expressed in terms of cents per vehicle-mile of traffic, were as follows: City streets, 0.78 cent; trunk-line roads, 1.05 cents; county roads, 2.37 cents; township roads, 1.29 cents.

THE results of a study of highway traffic upon township, county, and trunk-line highway systems and upon city streets within the State of Michigan are given in this report. The survey was conducted under a cooperative research agreement between the Bureau of Public Roads of the United States Department of Agriculture and the Highway Department of the State of Michigan.

The work was under the general supervision of E. W. James, Chief of the Division of Highway Transport of the Bureau of Public Roads, and Grover C. Dillman, State Highway Commissioner of Michigan. The project was directly in charge of L. E. Peabody, senior highway economist, assisted by C. B. Bishop, H. E. Cunningham, D. O'Flaherty, and L. S. Tuttle, all of the Division of Highway Transport.

OBJECTIVES OF THE SURVEY

The tendency of States to concentrate control of highways has greatly advanced within the last year. A system of dividing road construction or maintenance among small units of Government has resulted in inefficient use of road machinery, lack of financial ability to obtain technical direction and difficulty in coordinating highway improvements among adjacent governmental units. The interchange of traffic between township, county, and trunk-line systems and city streets produces a situation which requires the highest type of cooperation among administrative forces representing each system if efficient results are to be obtained.

Organization for the use of mass-production methods on secondary highways has taken various forms. In 1931 the Department of Highways of Pennsylvania assumed direct responsibility for more than 20,000 miles of secondary highway. Michigan legislation in the same year provided for gradual absorption of all township highways by the counties, with a financial grant to the counties from State highway funds, and with a measure of supervision of their expenditure by the highway department. North Carolina has placed all highways of the State under direct control of her highway commission. Virginia has made provision for the absorption of county highways by the State, effective July 1, 1932. A recommendation that county highways be taken over by the State has been made to the Governor of New York by a commission appointed to study the matter. Counties have taken over the entire township mileage in Iowa, and in Illinois township highway organizations are under the jurisdiction of the county superintendents of highways.

These administrative changes have resultant financial problems, the sound solution of which rests fundamentally upon the relative amounts and character of the traffic on the various classes of highway. Mutual demands for assistance among the various governmental units and conflicting opinions as to the amount of traffic interchanged rest at present upon hypothesis, or at best upon very fragmentary data. The relative responsibility of the several governmental units for the construction and maintenance of all of the highways within the State may be stated on the basis of facts obtained in extensive traffic surveys.

The Michigan transport survey is the first in which the traffic on all these highway systems—city streets, trunk lines, county highways, and township roads—has been simultaneously studied. Traffic of local and of nonlocal cars on each of these highway systems was observed at over 1,000 points covering approximately 4,000 sections of highway throughout a full year. In studying a township the use of trunk-line, county, and township roads of the township by cars owned within and outside of the township was obtained. In the case of the county, traffic use of the county and trunk-line systems by owners within and without the county was obtained.

Data as to use of city streets by local and nonlocal traffic were gathered at more than 400 points within the seven cities of Detroit, Grand Rapids, Flint, Lansing, Jackson, Ann Arbor, and Niles.

The primary objective of the survey was to obtain data indicative of the character and amount of use of the township, county, and trunk-line highways, and city streets.

A secondary objective was to obtain information on the tourist traffic in Michigan, its origin, the number of tourists and tourist cars, the length of stay, the mileage traveled, the types of accommodation used by tourists, and the value of tourist traffic to the State. It was also desired to determine the volume of city traffic dur-

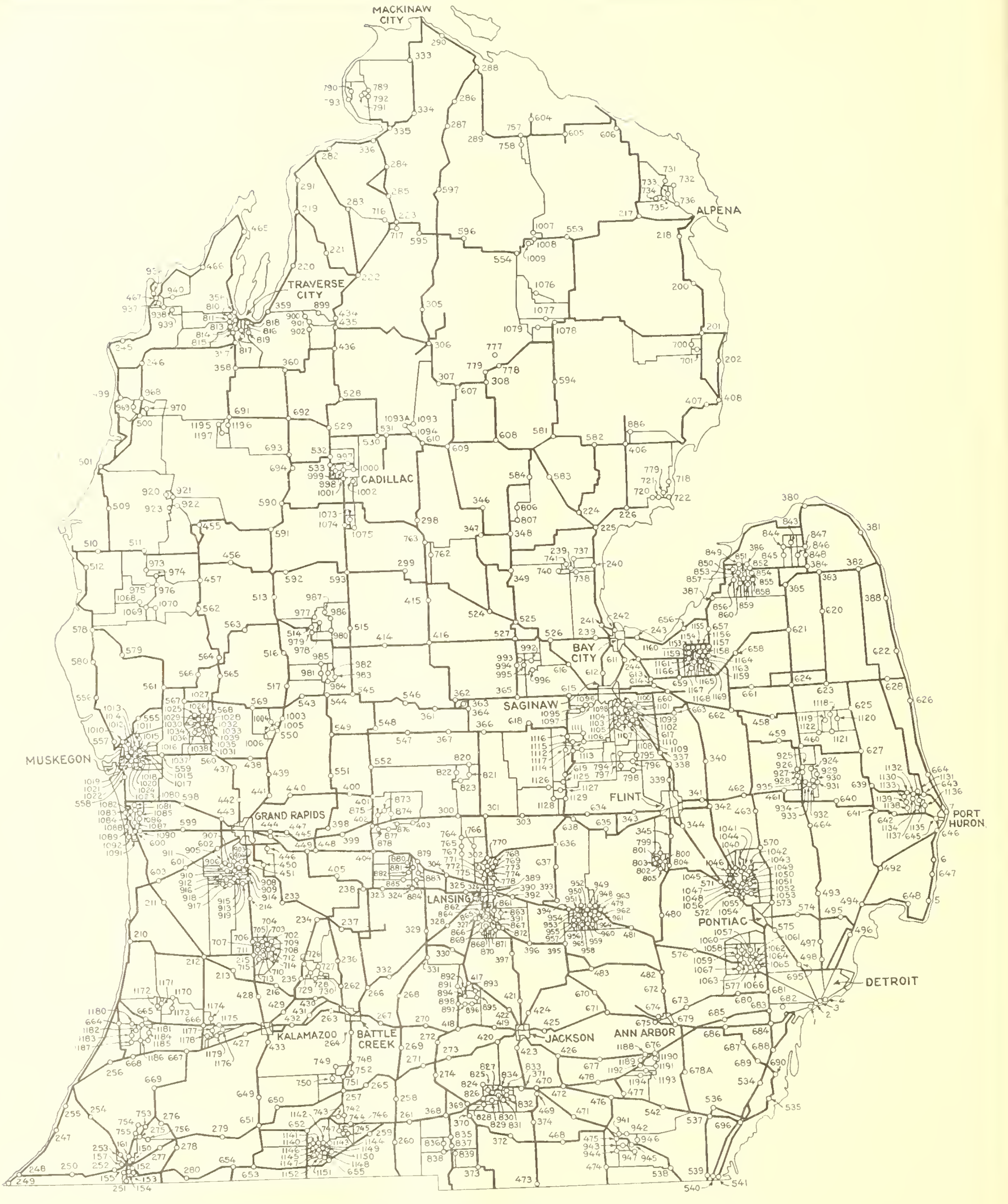

Figure 1.—Location of Traffic Stations in Southern Peninsula

ing peak hours of travel, total street widths, effective street widths after making deductions for parked vehicles, safety zones and other obstructions, and the presence of street-car tracks.

The material relating to tourist traffic appears separately in this issue (see p. 197). The detailed study of city traffic is omitted from this report because of lack of space.

SELECTION AND OPERATION OF TRAFFIC SURVEY STATIONS

Within the State of Michigan at the start of the survey were approximately 60,000 miles of township road, 17,000 miles of county highway, 8,000 miles of trunk line, and many thousand miles of city streets. The entire township road mileage was under the jurisdiction of 1,269 township units, while 83 counties administered their respective road systems, and the State highway department constructed and maintained all trunk-line highways.

It was impractical to operate traffic survey stations upon each mile, or even upon each section of this mileage. The townships were classified into 79 homogeneous groups according to the road mileage, types of roads, assessed valuation, and population. An average township within each group was selected as representative of the group and traffic stations were distributed within each such sample township so as to obtain adequate traffic data with respect to all classifications of highway, as well as all volumes of traffic. Stations were established at typical intersections of two township roads; a county road and a township road; two county roads; a trunk-line route with either a township or a county road; and at such additional points as to cover each route of travel within the township.

In each of the cities traffic survey stations were located at intersections where the data obtained would be representative of the traffic within the area. In general, these were located upon a cordon circumscribing the city, located at or near the city limits and controlling all of the major traffic routes. In addition, other stations were located within the interior of the city to obtain an accurate sample of all traffic movements within the area. These points were selected with the assistance of the city engineer or traffic engineer of the particular city. In the city of Detroit the stations adopted by the Rapid Transit Commission in their study of vehicular traffic and reported in 1930 were used.

A list of the townships comprising each group, with the "sample" township indicated, has been prepared in mimeograph form and may be obtained on request addressed to the Bureau of Public Roads. The location of the city stations is dealt with in the section of the report concerned with city traffic.

The information of major importance obtained at survey stations had reference to the origin of each vehicle. On all township roads, county roads, and trunk-line routes separately, each vehicle was placed in one of the following classes: Truck up to and including 1½ tons, truck over 1½ tons, passenger car, bus, or trailer. Each type of vehicle was further classified as to whether the owner lived within the township in which the survey station was located. If ownership was not in the township, the vehicle was classified either as owned within the county in which the station was located, elsewhere in the State of Michigan, or as a vehicle from outside the State. The "township," "county," and "State" vehicles, as defined in the foregoing, were further subdivided into those of urban ownership and those of rural ownership. Foreign vehicles were listed without regard to urban or rural ownership.

Collection of these data necessitated the stopping of all vehicles for questioning, or upon routes where traffic was too heavy to permit the stopping of vehicles, the taking of a complete density count of vehicles and a list of all registration tag numbers. These numbers were classified by means of the Michigan Department of State records showing residence of each licensee. The method of classification of vehicles at such stations was identical with that at the stations where it was possible to stop traffic. In all cases where it was impracticable to stop cars, a minimum of approximately 2,000 license tag numbers was secured during a 10-hour period and the results applied to the complete density count of vehicles over the same period. There resulted a minimum classification by means of tag numbers of 20 per cent of all vehicles passing the station and a 100 per cent classification at all but the heaviest traffic stations.

At the survey stations located within the cities vehicles were classified by means of the tag numbers as described above. The classification of vehicles was less elaborate at these stations, cars being separated into "city" and "noncity" classifications. A "city" or "local" car was defined as one owned within the city where the station was located, all other cars were classified as "noncity" or "nonlocal." In addition, traffic during the peak hour of travel was tallied separately at all city stations, and data were obtained for each adjacent street with regard to actual street width, effective street width, width of parking space, number of traffic lanes, street-car tracks, traffic lights, and other pertinent data about the intersection as related to traffic movement.

VOLUME AND ORIGIN OF TRAFFIC ON THE DIFFERENT CLASSES OF RURAL ROUTES

The location of the survey stations upon the rural routes of the southern peninsula is shown in Figure 1. Limitations in the size of the publication and the vast mileage of rural highway, totaling approximately 85,000 miles, prohibit the presentation of all of the rural road mileage. The maps show the location of the townships selected for intensive traffic analysis, the approximate location of the township stations within each such sample township, and the location of all stations upon the trunk-line routes. The number of all classes of vehicles observed during the period July, 1930, to July, 1931, upon the rural highways totaled nearly 12,000,000. Upon city streets more than 19,000,000 cars were counted and classified.

Figure 2 shows the flow of motor vehicles upon all trunk-line routes represented to scale. The data include average daily 24-hour traffic throughout the year and, in broken line, the average maximum daily traffic.

There is considerable monthly variation in traffic from the average monthly traffic of all types of vehicles. The traffic ranged from 64 per cent of the average in January to 148 per cent in August. The variation was greater for passenger cars—from 61 to 153 per cent—than for light trucks, with 79 per cent in January and 131 per cent in October. Bus traffic was more stable than that of any other type of vehicle, varying from 92 per cent in February to 114 per cent in October. Between local highways and trunk-line routes the differences in the range were small. The lightest volume of traffic on the trunk lines was in January, and in February on both county and township highways. On the respective systems the percentages were 63, 69, and 65 of the average monthly traffic. The high month on each system was August with 148 per cent for the trunk lines, and 150 and 144 per cent for county and township highways, respectively. There

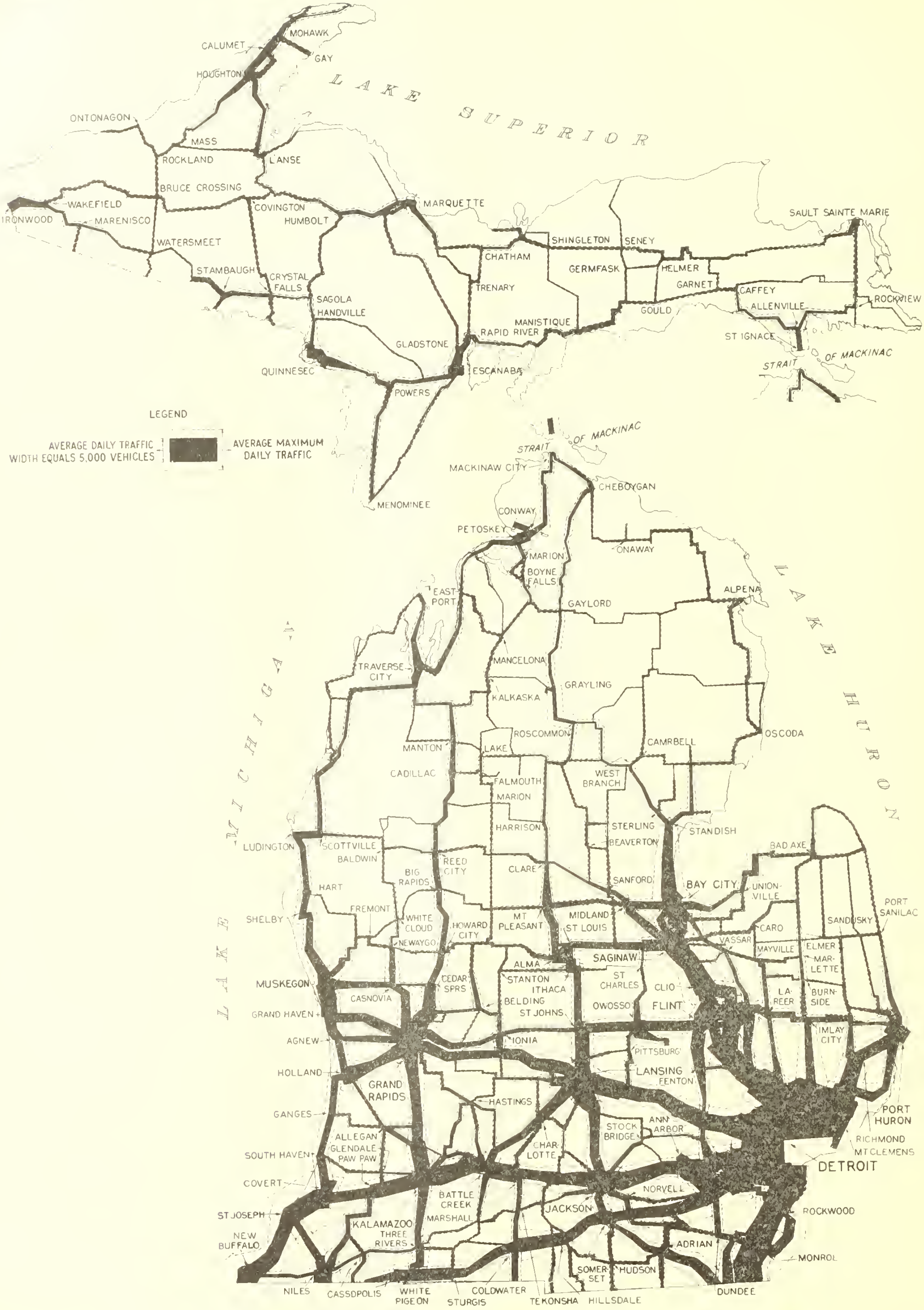

FIGURE 2.—AVERAGE DAILY TRAFFIC ON MICHIGAN TRUNK LINE HIGHWAYS, 1930–31

was great similarity in variation by days of the week on all three systems, Sunday being the heaviest day, with about 140 per cent of the average daily traffic.

Daily traffic averages within each sample township for each type of highway—township road improved and unimproved, county highways improved and unimproved, and trunk-line routes—separated by origin of the vehicle (within the township, within the county, within the State, and foreign) and for each type of vehicle (passenger cars, trucks up to and including 1½ tons, trucks over 1½ tons, trailers, and busses) were obtained from the station observations extending throughout the year. These data tabulated without separation by place of origin, may be obtained in mimeograph form from the Bureau of Public Roads. The traffic averages so obtained were applied to the total mileage of each type of highway in the corresponding township groups, resulting in an average daily vehicle-mileage by origin of vehicles upon the several political classes of highway. These daily vehicle-miles and the mileages of the several classes of highway within the township groups are tabulated in Table 1. The figures are summarized and corresponding percentages given in Table 2.

TABLE 1.—*Average daily vehicle-miles on township, county and trunk-line highways of township groups, classified according to township, county, or other origin*

Group of townships represented by—		Township roads in group	Travel on township roads from—			County roads in group	Travel on county roads from—			Trunk-line roads in group	Travel on trunk-line roads from—		
Township	County		Township	County	Other[1]		Township	County	Other[1]		Township	County	Other[1]
		Miles	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*	*Miles*	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*	*Miles*	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*
Ewing	Marquette	251.0	9,036	251	1,757	3.5	256	200	203	0.	------	------	------
Colfax	Oceana	311.1	1,822	1,565	1,627	1.4	102	80	81	5.3	975	1,378	3,78
Wilson, Bay de Noc	Kalkaska, Delta	710.8	7,940	1,606	772	62.9	929	895	580	0.	------	------	------
Elk	Lake	738.4	5,586	1,596	6,705	104.7	1,241	435	1,188	18.2	200	382	619
Gourley	Menominee	699.5	7,695	3,497	699	157.2	3,616	1,415	314	0.	------	------	------
Logan	Mason	854.0	3,322	1,350	1,599	176.5	2,462	3,580	1,588	22.9	710	2,931	5,725
Maple Ridge	Alpena	595.1	5,115	1,988	697	158.6	6,027	14,274	9,516	1.0	184	260	676
James	Saginaw	449.8	8,702	7,352	449	133.3	9,064	32,259	3,732	4.0	736	1,040	2,704
Friendship	Emmet	787.3	1,575	787	787	211.4	2,537	8,879	5,285	6.6	1,214	1,716	4,462
Whitney	Arenac	836.8	3,347	837	3,347	173.3	3,575	1,367	1,534	.3	55	78	203
Mount Forest	Bay	974.5	6,822	5,847	4,872	303.1	27,582	17,580	13,943	18.5	6,456	8,399	24,402
Flynn	Sanilac	1,197.4	4,790	3,592	1,197	388.3	17,085	18,250	20,192	17.2	1,858	2,683	3,646
Montrose	Genesee	1,126.1	22,522	7,883	3,378	617.9	42,635	64,880	46,960	10.7	535	10,657	20,266
Reading	Hillsdale	991.2	19,702	6,898	3,965	516.0	44,892	26,832	28,380	29.7	5,584	3,950	4,039
Hinton	Mecosta	616.0	3,610	934	934	186.4	6,710	3,728	8,761	6.9	1,270	1,794	4,664
Columbia	Van Buren	786.4	7,443	1,573	3,942	229.6	11,939	12,858	19,746	4.7	865	1,222	3,177
Riley	Clinton	1,253.3	18,800	8,773	5,013	532.1	20,220	17,559	18,091	9.3	484	2,046	6,017
Fulton	Gratiot	1,231.1	13,164	2,336	1,784	506.2	31,891	27,335	43,027	22.4	1,949	3,584	3,912
Carlson, Hudson	Gogebic, Mackinac	743.4	4,835	1,020	1,020	59.6	4,351	3,397	3,457	343.6	8,590	10,308	52,914
Secord	Gladwin	841.5	9,256	5,890	7,574	32.3	2,358	1,841	1,873	149.1	3,728	8,797	14,910
Franklin	Houghton	792.6	11,889	10,304	793	49.2	3,592	2,804	2,851	137.6	26,006	135,261	33,987
Southbranch	Crawford	485.4	485	485	485	95.2	95	190	476	156.3	2,032	2,188	22,820
Elmer, Manistique	Oscoda, Schoolcraft	706.6	2,120	707	707	98.9	3,659	989	692	115.0	21,725	15,065	25,760
Warner	Antrim	836.1	6,689	836	2,508	124.7	748	1,496	2,120	172.6	10,011	5,868	45,566
Lake	Roscommon	622.6	3,113	1,245	623	89.8	1,257	718	1,976	127.5	4,335	5,355	28,815
Briley	Montmorency	760.8	9,130	1,522	5,326	115.0	2,070	230	230	185.1	15,363	4,072	12,587
Forest	Cheboygan	707.4	1,415	0	1,415	110.9	6,498	2,694	4,383	103.5	19,768	16,560	39,744
Arvon	Baraga	850.5	4,919	4,157	1,606	103.1	1,650	1,546	619	159.6	11,491	5,905	3,192
Hatton	Clare	594.7	2,974	3,568	1,784	131.7	1,185	2,766	1,317	135.8	1,086	14,123	52,419
Reno	Iosco	815.9	7,343	2,448	3,264	142.9	2,429	857	1,143	147.6	27,158	38,376	99,778
Middle Branch	Osceola	1,077.1	4,840	2,331	1,078	166.8	2,335	2,502	1,334	137.4	3,847	8,519	24,182
Gustin	Alcona	682.1	4,775	1,364	682	138.1	2,072	552	276	154.5	7,570	12,669	21,476
Greenland	Ontonagon	881.0	12,334	881	881	200.6	38,716	6,219	4,614	154.6	10,049	6,648	20,716
Pleasanton, Bates	Manistee, Iron	824.2	7,498	5,450	1,250	199.3	6,378	10,962	3,388	154.1	9,708	32,053	48,696
Dafter, Springville	Chippewa, Wexford	635.1	3,342	635	635	249.7	14,483	26,468	12,235	100.7	11,782	7,855	9,264
Norway	Dickinson	1,478.5	22,178	2,957	1,478	407.4	10,592	2,852	815	237.2	251,906	96,778	91,085
Palmyra	Lenawee	780.9	11,284	6,848	1,845	280.3	23,265	72,598	28,310	112.6	12,836	76,230	131,855
Saint Charles	Saginaw	566.1	5,962	2,414	1,132	125.2	6,886	13,772	15,650	139.3	10,587	22,288	22,149
Martiney	Mecosta	751.6	5,178	2,756	2,088	120.4	2,649	3,612	2,528	79.5	1,192	7,950	12,164
Empire	Leelanau	717.7	2,992	1,314	2,032	163.6	4,090	4.254	9,816	128.3	4,106	3,336	7,698
Mattison	Branch	1,110.0	12,210	6,660	4,440	261.0	6,003	10,179	8,091	145.6	5,533	8,736	19,947
Almena	Van Buren	1,100.1	16,501	6,600	7,086	223.5	20,940	5,797	18,484	145.8	7,727	22,891	34,992
Riverside	Missaukee	1,033.1	5,821	1,251	1,907	203.8	4,484	2,038	2,242	122.1	5,372	3,175	14,652
Mount Haley	Midland	1,060.6	10,606	4,516	3,181	270.8	5,958	3,250	3,791	114.8	2,066	4,822	12,398
Pierson	Montcalm	833.1	12,477	3,269	22,004	399.4	8,787	10,384	8,787	135.0	12,420	19,170	157,680
Barry	Barry	964.6	18,327	3,858	10,821	309.7	25,705	11,769	37,783	137.1	16,863	21,388	74,034
Athens	Calhoun	1,014.5	10,145	3,044	2,029	287.5	12,075	5,750	7,475	132.3	19,051	31,752	27,386
Pokagon	Cass	1,140.0	19,380	10,260	5,700	269.2	8,076	6,461	9,960	186.3	13,600	56,263	153,884
Rush	Shiawassee	973.6	13,738	2,948	1,473	379.7	20,884	18,985	7,974	117.4	19,019	59,404	72,084
Ionia	Ionia	932.1	16,066	4,749	2,234	340.5	91,594	30,645	36,774	141.1	79,016	31,606	101,874
Springport	Jackson	788.3	11,129	3,200	5,565	253.7	14,461	8,880	13,700	139.6	9,353	8,655	22,476
Fenton	Genesee	827.9	37,150	17,762	7,240	349.2	60,062	49,237	36,666	122.8	78,838	112,485	64,470
Saline	Washtenaw	965.5	19,310	5,793	4,828	292.5	28,958	8,190	18,428	160.2	21,627	35,885	232,931
Bangor	Van Buren	760.8	13,491	3,057	2,554	350.1	27,308	19,256	21,006	172.4	27,929	35,514	32,411
De Witt	Clinton	897.2	22,209	3,775	14,862	323.0	23,256	16,796	51,034	153.0	27,999	49,572	302,634
Farmington	Oakland	607.1	21,154	4,228	13,920	423.5	59,290	61,408	158,812	163.4	89,707	66,014	754,908
Clyde	Saint Clair	879.6	6,418	5,050	880	223.5	3,800	6,258	1,311	106.5	5,644	30,140	16,294
Moscow	Hillsdale	956.5	8,927	5,101	2,550	180.2	7,568	8,289	11,172	83.6	5,131	19,228	168,621
Goodland	Lapeer	651.0	7,372	2,788	3,277	238.0	8,806	6,188	12,376	79.6	2,866	5,890	47,282
Garfield	Grand Traverse	740.2	21,237	4,615	4,980	195.1	31,606	10,730	15,023	105.2	60,806	19,778	97,100
Grand Haven	Ottawa	1,453.6	23,667	2,906	8,184	315.4	105,559	33,132	38,479	169.2	58,205	38,916	191,627
Danby	Ionia	834.5	13,414	4,173	6,250	258.1	11,873	15,744	17,035	104.0	5,200	31,736	228,384
Burr Oak	St. Joseph	939.4	13,098	5,438	2,249	271.4	16,013	9,499	10,042	99.3	7,745	27,010	147,163
Ashland	Newaygo	873.5	11,874	1,582	3,163	276.4	14,373	3,593	6,634	118.5	24,530	17,894	97,881
Martin	Allegan	969.7	26,117	5,098	4,286	314.8	41,239	8,185	17,314	76.6	16,163	17,924	109,232
Fairgrove	Tuscola	780.6	16,025	3,357	1,289	206.7	16,123	10,542	6,408	99.2	13,690	12,896	15,178
Bridgeport	Saginaw	868.6	9,878	6,975	1,737	276.5	16,590	37,604	14,931	108.3	15,920	134,406	269,992
Independence	Oakland	665.3	9,248	4,657	1,996	233.8	18,470	19,172	12,625	138.7	17,199	66,853	266,443
Handy	Livingston	652.3	13,072	1,868	2,801	270.3	21,354	22,165	11,082	106.3	23,173	16,476	276,061
Winsor	Huron	677.0	19,760	7,105	2,165	314.5	35,224	32,708	13,838	115.4	31,850	30,696	20,310
Byron	Kent	665.6	21,483	6,974	3,407	279.6	45,556	44,712	31,303	123.6	8,776	133,982	157,096
Delhi	Ingham	594.1	14,726	8,270	1,595	296.3	25,778	36,149	8,296	133.4	71,636	247,857	155,278
Muskegon	Muskegon	440.0	9,210	440	440	420.6	61,828	5,047	3,785	125.4	136,310	44,642	65,459
Total		60,214.0	803,814	279,164	244,823	17,175.4	1,273,722	994,467	995,918	7,690.9	1,442,218	2,044,974	5,317,464

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[1] Includes vehicles classified as "State" and "Foreign."

Table 2.—*Traffic upon township, county, and trunk-line highway systems originating within the township, within the county, or elsewhere*

Road type	Origin	Daily vehicle-miles	Percentage of total	Percentage of total traffic on rural roads
Township road	Township	803, 814	60. 5	9. 9
	County	279, 164	21. 0	
	Other [1]	244, 823	18. 5	
		1, 327, 801	100. 0	
County road	Township	1, 273, 722	39. 0	21. 4
	County	994, 467	30. 5	
	Other [1]	995, 918	30. 5	
		3, 264, 107	100. 0	
Trunk line	Township	1, 442, 218	16. 4	65. 7
	County	2, 044, 974	23. 2	
	Other [1]	5, 317, 464	60. 4	
		8, 804, 656	100. 0	
Total rural roads		13, 396, 564		100. 0

[1] Includes vehicles from outside the State of Michigan.

Traffic on rural roads totals 13,400,000 vehicle-miles per day. Sixty-six per cent of this traffic is carried by the trunk-line routes, which constitute 9 per cent of the rural mileage; 24 per cent of the traffic is carried by the county roads, which are 20 per cent of the rural mileage. The township highways carry but 10 per cent of the rural road traffic, although the mileage of township roads is 71 per cent of all rural mileage. The distribution of traffic by origin of vehicles varies widely between the three highway systems. Upon the township roads more than 60 per cent of the traffic originates within the local township and more than 81 per cent within the local county, while upon trunk-line routes more than 80 per cent of the traffic originates at points outside the township in which the station is located, and more than 60 per cent at points outside the county. Traffic volumes per day averaged 22 vehicles upon the township roads, 190 vehicles on county roads and 1,144 vehicles upon the trunk-line routes.

Township roads are a relatively small factor in the total traffic movement, producing but one-tenth of the total vehicle-miles, with more than 60 per cent of the township road usage originating within the township.

USE OF RURAL ROADS BY MICHIGAN VEHICLES OF CITY AND RURAL OWNERSHIP

It will be recalled that cars originating at points outside Michigan were classified by field recorders simply as "foreign" without separation into vehicles of city or rural ownership. All cars bearing Michigan tags were classified with respect to city or rural ownership.

Using the method previously described for obtaining traffic volumes by unit of origin, data for Michigan vehicles divided between cars owned within the cities and those rurally owned are tabulated in Table 3. These data are further condensed for convenience in Table 4.

Two-thirds of the usage of township roads is by cars owned rurally, and this ratio declines to 44 per cent upon county roads and to 19 per cent upon the trunk-line system. City-owned cars produce nearly 70 per cent of the traffic upon the rural highway system, and nearly 75 per cent of their travel is upon the trunk-line system.

Approximately 5 per cent of the travel of city-owned cars is upon the township road system. But a small portion of the average trip of a city-owned car is over the township roads, and the use of county highways by the city dweller is about four times as great as his use of the township roads.

USE OF RURAL HIGHWAYS BY FOREIGN TRAFFIC

Motor vehicles from States other than Michigan use the trunk-line routes of the rural highways almost exclusively. Foreign vehicles produce a total of 1,115,752 vehicle-miles per day upon the rural highway systems. Nearly 86 per cent of all foreign traffic is carried by the trunk-line system, and approximately one per cent of the foreign traffic is carried by the township roads of Michigan. Detailed figures are presented in Table 5.

Slightly over 8 per cent of all travel upon the rural highways is by foreign vehicles. They constitute 10.8 per cent of the use on trunk lines, 4.5 per cent upon the county highways, and 1.1 per cent on township roads.

LOCAL AND NONLOCAL TRAFFIC ON CITY STREETS

The total use of city streets, expressed in vehicle-miles, was developed from the figures of gasoline consumption of the State. These figures indicate a total consumption of 725,386,562 gallons from July 15, 1930, to July 15, 1931, the dates of the survey. Replies to more than 5,000 questionnaires, distributed to motorists throughout the State during the period of the survey, in connection with a study of the highway finance of the State, give an average travel of 13.4 miles per gallon for all types of vehicle. Applying this average mileage per gallon to the indicated consumption within Michigan and reducing the resulting quantity to a daily basis, the average daily travel upon all rural roads and city streets is 26,600,000 vehicle-miles. From this figure may be taken the average daily vehicle mileage upon rural roads, 13,400,000, leaving an average daily use of city streets of 13,200,000 vehicle-miles.

Traffic in each city is composed of two important elements—the movement of vehicles owned within the city, or local traffic, and the movement of vehicles owned elsewhere, or nonlocal traffic.

In order to effect this separation of city traffic, 7 cities ranging in population from 10,000 to more than 1,500,000 were selected for field observation and classification of the traffic moving within them. The sample cities were Detroit, Grand Rapids, Flint, Lansing, Jackson, Ann Arbor, and Niles. A complete statement of detailed methods and results of this study is published in an appendix. Briefly, the city traffic survey involved a selection of more than 400 stations at which license tag numbers were recorded and later classified as to residence of the owners by reference to registration records. In the determination of the percentage of local use, the data collected on through streets and on local streets were analyzed separately. The percentages of local and nonlocal use in each city were computed separately for through and local streets and the results weighted by the relative mileages of each class of city streets. The results are shown in Table 6.

Examination of this table discloses that there is a uniform increase in the percentage of local traffic with increase in the size of city, both on through and nonthrough streets; that the percentage of local traffic on nonthrough streets is considerably in excess of the percentage upon through streets and that the differentials, in percentages of local use, between through and nonthrough streets are considerably lessened as the size of the city increases.

TABLE 3.—*Average daily vehicle-miles on township, county, and trunk-line roads of township groups classified according to rural or city origin*

Group of townships represented by—		Township roads		County roads		Trunk-line roads	
Township	County	Rural origin	City origin	Rural origin	City origin	Rural origin	City origin
		Vehicle-miles	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*
Ewing	Marquette	10,542	251	293	366	----	----
Colfax	Oceana	3,422	1,556	117	146	1,104	4,832
Wilson, Bay de Noc	Kalkaska, Delta	9,210	1,422	1,321	1,152		--
Elk	Lake	11,076	3,692	1,887	1,047	528	657
Gourley	Menominee	10,492	700	4,873	314	-	----
Logan	Mason	10,248	1,708	5,472	2,294	2,450	6,229
Maple Ridge	Alpena	10,117	3,571	14,133	16,970	242	1,062
James	Saginaw	10,795	5,847	16,529	28,260	855	1,647
Friendship	Emmet	1,575	787	6,342	8,667	1,375	6,017
Whitney	Arenac	4,184	2,510	3,986	1,560	62	274
Mount Forest	Bay	9,745	6,822	28,188	30,310	1,329	[illegible]
Flynn	Sanilac	5,987	3,592	24,463	30,676	2,408	5,076
Montrose	Genesee	23,649	10,135	47,578	105,013	4,334	25,937
Reading	Hillsdale	19,824	11,891	47,987	44,376	4,663	7,663
Hinton	Mecosta	4,312	1,232	10,252	8,761	1,437	6,291
Columbia	Van Buren	6,291	2,359	11,235	25,486	979	4,285
Riley	Clinton	23,813	7,520	34,054	22,348	1,767	6,333
Fulton	Gratiot	14,773	3,693	13,027	[illegible]	4,010	[illegible]
Carlson, Hudson	Gogebic, Mackinac	5,204	715	4,975	6,230	20,616	[illegible]
Secord	Gladwin	12,622	10,098	2,696	3,376	8,797	[illegible]
Franklin	Houghton	18,230	3,963	4,107	5,143	68,387	[illegible]
South Branch	Crawford	485	485	190	476	4,220	21,277
Elmer, Manistique	Oscoda, Schoolcraft	2,826	707	2,571	2,670	15,870	43,700
Warner	Antrim	7,525	1,672	2,743	1,621	15,189	42,805
Lake	Roscommon	4,981	623	2,155	1,616	11,985	24,098
Briley	Montmorency	11,412	4,565	2,300	230	20,361	10,771
Forest	Cheboygan	2,122	707	2,551	1,885	11,552	24,736
Arvon	Baraga	14,458	4,252	1,856	1,650	8,938	11,172
Hatton	Clare	4,758	2,971	5,161	1,976	7,741	52,419
Reno	Iosco	10,607	4,080	5,287	1,000	11,218	12,676
Middle Branch	Osceola	6,463	3,231	3,169	3,002	6,870	28,717
Gustin	Alcona	5,457	1,364	2,348	411	12,514	[illegible]
Greenland	Ontonagon	12,334	881	44,533	4,012	18,707	11,842
Pleasanton, Bates	Manistee, Iron	11,539	8,242	9,766	10,164	16,797	[illegible]
Dafter, Springville	Chippewa, Wexford	3,811	637	23,222	26,718	10,271	15,500
Norway	Dickinson	17,742	7,392	8,448	5,704	126,664	275,625
Palmyra	Lenawee	46,399	6,247	47,371	65,310	31,753	91,954
Saint Charles	Saginaw	6,793	3,397	10,016	25,290	11,840	40,974
Martiney	Mecosta	6,764	2,255	5,057	3,492	7,076	12,561
Empire	Leelanau	3,588	1,435	6,871	8,507	5,152	7,955
Matteson	Branch	15,510	6,660	7,569	7,569	10,483	21,112
Almena	Van Buren	20,902	8,801	11,304	12,740	14,143	45,927
Riverside	Missaukee	5,166	2,066	5,706	3,057	7,682	15,507
Mount Haley	Midland	13,788	4,242	8,395	4,604	5,970	12,743
Pierson	Montcalm	15,829	29,158	12,781	14,778	24,765	[illegible]
Barry	Barry	21,221	6,752	41,840	31,589	37,702	67,642
Athens	Calhoun	10,145	6,087	13,860	10,925	24,433	53,449
Pokagon	Cass	21,660	12,510	10,230	11,037	22,729	[illegible]
Rush	Shiawassee	12,657	2,921	28,478	19,365	36,511	111,660
Ionia	Ionia	17,710	7,457	59,928	96,702	40,072	164,382
Springport	Jackson	11,824	4,730	16,237	19,535	42,983	25,547
Fenton	Genesee	13,246	19,043	31,079	113,141	37,686	210,848
Saline	Washtenaw	18,344	10,620	26,940	27,788	28,996	88,395
Bangor	Van Buren	13,694	6,847	23,457	36,760	16,895	76,512
De Witt	Clinton	24,224	16,150	39,729	49,742	53,856	362,787
Farmington	Oakland	24,285	17,606	73,688	197,351	114,280	772,227
Clyde	Saint Clair	8,796	4,398	6,258	5,140	15,868	35,358
Moscow	Hillsdale	11,478	2,870	13,335	12,974	15,048	114,281
Goodland	Lapeer	11,772	4,578	13,090	13,864	12,020	43,664
Garfield	Grand Traverse	15,544	19,985	21,851	33,362	39,629	121,927
Grand Haven	Ottawa	15,983	15,983	53,618	110,765	31,640	221,652
Danby	Ionia	18,359	7,510	24,603	19,874	19,552	[illegible]
Burr Oak	Saint Joseph	11,273	8,455	11,927	18,184	11,916	162,875
Ashland	Newaygo	12,229	6,988	13,267	10,780	19,078	116,130
Martin	Allegan	31,030	7,758	47,220	16,684	22,903	105,685
Fairgrove	Tuscola	14,054	5,464	47,156	15,769	17,459	21,665
Bridgeport	Saginaw	14,292	6,080	29,369	39,263	50,793	173,816
Independence	Oakland	8,649	5,322	17,067	30,394	40,362	299,515
Handy	Livingston	13,046	3,911	35,680	18,410	19,134	283,562
Winsor	Huron	18,279	6,093	42,158	38,684	76,120	46,645
Byron	Kent	25,293	6,656	64,308	53,401	22,742	257,779
Delhi	Ingham	15,447	10,694	31,764	38,223	55,228	390,195
Muskegon	Muskegon	4,840	5,720	27,339	35,330	59,440	178,444
Total		883,801	429,387	1,384,819	1,733,256	1,456,417	6,393,132

TABLE 4.—*Traffic of Michigan vehicles on rural highways by class of highway and situs of ownership*

Highway type	Situs of ownership	Daily vehicle-miles	Percentage
Township road	City	429,387	32.7
	Rural	883,801	67.3
County road	City	1,733,256	55.6
	Rural	1,384,819	44.4
Trunk line	City	6,393,132	81.4
	Rural	1,456,417	18.6
Total rural roads	City	8,555,775	69.7
	Rural	3,725,037	30.3

TABLE 5.—*Daily use of rural highways by Michigan and non-Michigan vehicles*

Highway type	Daily vehicle-miles					Percentage of foreign vehicle-miles
	Foreign vehicles	Percentage	Michigan vehicles	Percentage	Total	
Township road	14,613	1.3	1,313,188	10.7	1,327,801	1.1
County road	146,032	13.1	3,118,075	25.4	3,261,107	4.5
Trunk line	955,107	85.6	7,849,549	63.9	8,804,656	10.8
Total	1,115,752	100.0	12,280,812	100.0	13,396,561	8.3

TABLE 6.—*Origin of traffic on through and local streets in seven Michigan cities*

City and type of street	Percentage of street mileage	Origin of traffic		
		Local	Non-local [1]	Non-Mich.
		Per cent	*Per cent*	*Per cent*
Niles (population, 11,326).				
Through	13.1	41.2	22.1	36.7
Nonthrough	86.9	60.5	19.0	20.5
Weighted average		58.0	19.4	22.6
Ann Arbor (population, 26,944).				
Through	11.9	45.6	46.3	8.1
Nonthrough	88.1	64.0	29.9	6.1
Weighted average		61.8	31.9	6.3
Jackson (population, 55,187).				
Through	7.5	60.5	30.4	9.1
Nonthrough	92.5	73.8	23.0	3.2
Weighted average		72.8	23.6	3.6
Lansing (population, 78,397).				
Through	8.9	56.4	37.6	6.0
Nonthrough	91.1	70.6	26.5	2.9
Weighted average		69.3	27.5	3.2
Flint (population, 156,492).				
Through	5.9	66.3	30.3	3.4
Nonthrough	94.1	80.4	17.7	1.9
Weighted average		79.6	18.4	2.0
Grand Rapids (population, 168,592).				
Through	6.5	72.8	21.9	5.3
Nonthrough	93.5	80.9	16.6	2.5
Weighted average		80.4	16.9	2.7
Detroit (population, 1,568,662).				
Through	3.2	73.1	22.4	4.5
Nonthrough	96.8	81.1	15.2	3.7
Weighted average		80.9	15.4	3.7

[1] Exclusive of traffic from outside the State.

The seven cities contain approximately 63 per cent of the Michigan urban population and it was necessary to estimate the percentages of local and nonlocal use for the remaining 37 per cent upon the basis of the data obtained in the seven cities. This was done by examining the relationship between the population of the cities and the percentage of local use and, as a check, establishing the relationship between the ratio of through streets to total street mileage of each city and the percentage of local use. Both of these relationships are fully discussed in the appendix. Combining the estimates of local use obtained by these relationships with the percentages observed in the seven sample cities resulted in an average local use in all cities of Michigan amounting to 69 per cent of the total urban traffic of the State. A separation of the total urban traffic of 13,200,000 daily vehicle-miles upon this basis, results in local use of city streets of 9,100,000 daily vehicle-miles and nonlocal use of 4,100,000 daily vehicle-miles.

TRAFFIC DATA SUMMARIZED

As a summary of the facts brought out in the preceding pages, the data on rural roads in Table 2 are repeated in Table 7, and combined with the figures obtained for traffic on city streets, so as to show the relative distribution of local and non-local traffic on the different classes of highway.

The predominantly local character of the township roads is indicated by the 60.5 per cent of traffic on these roads which originates within the township, while only 18.5 per cent comes from outside the county. Similarly 69.5 per cent of the traffic on county roads is by vehicles originating within the county. That the trunk-line roads are essentially arteries of through traffic is demonstrated by the 60.4 per cent of the traffic on these roads originating outside of the county.

TABLE 7.—*Traffic on each class of highway in Michigan distributed according to origin or character*

Road type	Origin or character	Daily vehicle-miles	Percentage of total for type	Percentage of total State traffic
Township roads	Township	803,814	60.5	5.0
	County	279,164	21.0	
	Other	244,823	18.5	
		1,327,801	100.0	
County roads	Township	1,273,722	39.0	12.2
	County	994,467	30.5	
	Other	995,918	30.5	
		3,264,107	100.0	
Trunk lines	Township	1,442,218	16.4	33.1
	County	2,044,974	23.2	
	Other	5,317,464	60.4	
		8,804,656	100.0	
All rural roads		13,396,564		50.3
City streets	Local	9,131,388	69.0	49.7
	Nonlocal	4,102,507	31.0	
		13,233,895	100.0	
Total State traffic		26,630,459		100.0

Table 4 shows that 81.4 per cent of the traffic of Michigan vehicles on the trunk-line roads, or 6,393,132 daily vehicle-miles, is of city origin. The total population of incorporated areas in Michigan in 1930 was 3,596,394, or 74 per cent of Michigan's population of 4,842,325. These facts indicate that residents of cities use the trunk-line roads to a slightly greater extent than the inhabitants of rural areas. However, the percentages are not greatly different, and they further emphasize the general character of the trunk-line roads.

On the city streets 31 per cent of the traffic, amounting to about 4,100,000 vehicle-miles per day, is of nonlocal origin. Part of this traffic is by vehicles from other cities and part by vehicles of rural origin. It would be useless to estimate these fractions, although the predominance of city traffic on the trunk lines suggests that the greater portion of the nonlocal city traffic comes from other cities. The traffic of city vehicles on township and county roads is shown in Table 4 to be 2,162,643 vehicle-miles per day, which is approximately half of the nonlocal traffic on city streets. It seems probable from these facts that the use of city streets by rural residents is not greatly different from the use of local rural roads by city residents, while the use of the trunk lines by the two classes of residents is approximately in the proportion of urban and rural population.

STREET AND HIGHWAY EXPENDITURES IN MICHIGAN

Financial data relating to highway expenditures in Michigan are not available for a period concurrent with the period of the traffic survey, although actual expenditures for that period may be closely approximated by available data. The financial statement of the State Highway Department covers the period July 1, 1930, to June 30, 1931, inclusive. Corresponding data for the townships, counties and cities are available for the calendar year 1930.

The records of the State Highway Department indicate total disbursements for the above-named period amounting to $45,582,894. Of this total, $10,622,880 represents direct payments of registration fees to the counties, and $1,060,014 represents direct payments to cities for maintaining and widening trunk-lines through the cities, leaving a net total of $33,900,000 for construction, maintenance and overhead of the trunk-line highway system.

Replies to questionnaires sent to the counties give total receipts for highway purposes in the year 1930 of $32,376,706, and highway expenditures during the same year amounting to $28,185,300.

In the financial survey previously referred to, highway expenditures by local units covering the year 1930 were obtained. These figures show the expenditures for highways by townships and by five groups of incorporated places. They do not include the expenditures by the State, including Federal aid, or by the counties. They are strictly local expenditures made by the townships and the incorporated places under their own direction, and they include expenditures made and assessed against abutting property. These latter expenditures have not been usually included in the highway and street expenditures as commonly totaled for a State, so that the figures for Michigan are very much larger than those commonly published, but it has been assumed that assessments for street improvements, or rather expenditures for street improvements to be later collected from abutting property, are highway expenditures for that year within the meaning of the term.

A recapitulation of the net expenditures for highway purposes by units of Government is as stated in Table 8.

TABLE 8.—*Expenditures for highways by units of government during a 1-year period in 1930–31*

Governmental unit:		
State	$45, 582, 894	
Less payments to counties	10, 622, 880	
Less payments to cities	1, 060, 014	
		$33, 900, 000
Counties		28, 185, 300
Townships		6, 264, 384
Cities		37, 822, 997
Total		106, 172, 681

The expenditures listed in Table 8 are not annual highway costs, as they include capital expenditures as well as current items. Payments of bond principal are not included. State Highway Department expenditures during the years 1925 to 1929 ranged from 22 millions to 34 millions of dollars and averaged $26,600,000 per year. Expenditures for local highways during this period ranged from 21 millions to 43 millions of dollars and averaged $34,200,000 per year.

On the basis of the expenditures shown in Table 8 and the total annual traffic on each system, the expenditures per vehicle-mile on each of the highway systems of the State, including city streets, were computed and are as given in Table 9. It will be observed that the average for all highways of the State, including city streets, is 1.18 cents per vehicle-mile; and that the figures vary from 0.78 cent for city streets to 2.37 cents for county roads.

TABLE 9.—*Expenditures per vehicle-mile in 1930–31 on the several highway systems of Michigan*

Highway system	Expenditures per vehicle-mile	Annual vehicle-miles on system [1]
	Cents	
Township roads	1. 29	484, 647, 365
County roads	2. 37	1, 191, 399, 055
Trunk lines	1. 05	3, 213, 699, 440
City streets	. 78	4, 830, 371, 675
Average, all highways	1. 18	----

[1] Based on Table 7.

These expenditure figures, although they can not be regarded as true annual costs, offer a basis of comparison with the reduction in cost per vehicle-mile in the operation of motor vehicles, effected by the improvement of roads. Data on this subject were developed in experiments by Prof. T. R. Agg, and reported in Bulletin 69 of the Engineering Experiment Station, Iowa State College, in 1924. Professor Agg's figures indicate that the average cost of operating an automobile over a high-type surface such as concrete, brick, or asphalt is approximately 2½ cents per mile less than the cost of operating over an ordinary earth road. A low-type surface such as gravel reduces operating costs about 1 cent per vehicle-mile; an intermediate surface, such as bituminous macadam, about 2 cents. A well-packed earth road, as opposed to average or ordinary earth roads, was shown to effect a reduction of about one-half cent per vehicle-mile.

From these figures it is evident that the expenditure of 0.78 cent per vehicle-mile on city streets, with a high type of improvement and dense traffic, is more than justified by the reduction in cost of operation of motor vehicles. The same is true of the trunk-line roads, which have an average traffic density of 1,144 vehicles per day. On the city streets and trunk-line highways the expenditures listed are chiefly for construction, maintenance playing a relatively small part in the total.

The county roads, which are mainly of intermediate and low types, and on which maintenance expenditures become more important, present a different situation. The expenditures per vehicle-mile amount to 2.37 cents. The average density of traffic on the county roads is 190 vehicles per day. This rate of expenditure can hardly be justified from the standpoint of economy in motor vehicle operation; and the fact that expenditures per vehicle-mile on county roads are conspicuously higher than on any of the three other classes of highway suggests that they may be excessive. It may be that the mileage improved has been overextended or that the type of improvement has been too expensive, or that a combination of these two factors has led to the high rate. In this connection it is noted that in Michigan, according to the latest available figures, there are 77,389 miles of local road, of which 33,408 miles, or 43.2 per cent, are surfaced. More than 97 per cent of the county highways are surfaced, and 27 per cent of the township highways are improved with gravel surface or a higher type. The surfaced local mileage for the United States as a whole in 1930 was but 17.5 per cent of the total.

It must be recognized that the county roads, on which 39 per cent of the traffic is of local (i. e. township) origin, render services, both social and economic, which can not be computed in terms of reduced transportation costs. However, there is a definite indication of overexpenditure; and plans for further development of the county system should take into account the extent to which the traffic justifies the outlay.

LOW TRAFFIC DENSITIES FOUND ON TOWNSHIP ROADS

Table 9 shows that the expenditures on the township roads in 1930–31 amounted to 1.29 cents per vehicle mile. This figure can not very well be compared with reduction in cost of motor vehicle operation. Seventy-three per cent of the township mileage is unimproved, and corresponds to ordinary earth road, the lowest type considered in Professor Agg's figures, quoted above. It is doubtful if the expenditures on township roads could be said to produce a reduction of more than one-half cent per vehicle-mile in transportation costs.

The great bulk of expenditures on these roads is for maintenance, nearly all in the case of the unimproved roads. Reliable information as to the actual maintenance cost of township highways is difficult to obtain and but little has been written about this phase of highway economics.

Dividing the expenditure on township roads, as given in Table 8, $6,264,384, by the total mileage in the system, 60,214, we obtain $104 as the average expenditure per mile. This figure includes construction as well as maintenance costs. Township highway expenditures for the year 1930 were obtained from the State authorities for 78 townships. The reported expenditures for all these townships also averaged $104 per mile. The figures varied from very low values to over $500 per mile. Those townships in which the township highways were all unimproved (except for 1 mile) reported an average expenditure of $51 per mile. This figure is very near the minimum which could be expended with profit in maintaining an unimproved road.

The observed average daily traffic on township roads varied from less than one to 279 vehicles per day, the average being 22, and the median 20. Table 10 shows a percentage distribution of those township highways on which observations were made, on the basis of traffic density. The corresponding mileages, based on all the township mileage in the State, are also given. Thus we see that over half the mileage supports a traffic of 20 vehicles per day or less; about one-fourth has a traffic of 10 or less; and on nearly 14 per cent, or over 8,000 miles, the traffic is no more than 5 vehicles per day. If we apply the maintenance figure of $51 per mile to some of these low densities we obtain the following figures:

TABLE 10.—*Cumulative percentage of observed township mileage having traffic densities from 1 to 279 vehicles per day and corresponding mileages computed by applying these percentages to the total of township highway mileage in the State*

Average daily traffic density	Percentage of observed township mileage	Corresponding mileage based on all township mileage in State	Average daily traffic density	Percentage of observed township mileage	Corresponding mileage based on all township mileage in State
5 or less	13.58	8,177	40 or less	80.45	48,442
10 or less	25.51	15,361	45 or less	86.09	51,838
15 or less	39.58	23,833	50 or less	88.46	53,265
20 or less	51.90	31,251	75 or less	95.97	57,787
25 or less	63.16	38,031	100 or less	97.93	58,968
30 or less	70.73	42,589	150 or less	99.11	59,678
35 or less	76.65	46,154	279 or less	100.00	60,214

Expenditures per vehicle-mile

Vehicles per day:	Cents
1	13.97
5	2.80
10	1.40
20	.70

To set a limit, in terms of cents per vehicle-mile, on the expenditures to be made on such roads, would be an arbitrary procedure. It is clear, however, that a traffic of 1, 5, 10, or even more vehicles per day is insufficient for the reduction in transportation costs to pay the cost of maintenance. There are considerations more or less intangible which may justify high expenditures per vehicle-mile on very lightly traveled township roads. The State may be said to have a certain obligation, in connection with public education and the general welfare, to provide access to the land and homes of its citizens. This principle can not, of course, be pushed to the limit of building a road to the remotest habitation. The obligation exists none the less; and it would be well to recognize that a considerable portion of the expenditure on local roads must be justified by the general social and economic benefits to be derived.

APPENDIX

METHOD OF DETERMINING LOCAL TRAFFIC IN MICHIGAN CITIES

The use of city streets by residents and nonresidents was determined for the seven cities of Detroit, Grand Rapids, Flint, Lansing, Jackson, Ann Arbor, and Niles. These cities are representative of the population range from 10,000 to more than 1,500,000 and data obtained in them are applicable to other Michigan cities within this range.

Forty-three per cent of the passenger cars and 36 per cent of the trucks registered in Michigan in 1930 were registered in these seven cities. Their population in 1930 was 43 per cent of the population of the State, 57 per cent of the population of the 475 incorporated places in Michigan, and 63 per cent of the urban population of Michigan. Within these cities were 71 per cent of the registered motor vehicles of all cities with a population of 10,000 or over in 1930, and 70 per cent of the population of such cities.

A typical distribution of traffic stations is illustrated in the map of Flint, Figure 1. The stations are well distributed throughout the city and are sufficient in number to obtain data relative to all traffic movements within the city. All important sources of traffic were covered by stations. The trunk lines passing through Flint are shown by heavy lines on the map. The distribution of stations is similar to that in all of the cities except Detroit. In that city traffic stations were located on three cordons.

The inner cordon covered all intersections on Grand Boulevard; the middle cordon was on Artillery, Livernois, Davison, Six Mile Road, and Conners Streets; the outer cordon was on Eight Mile Road and at the crossings of the River Rouge near the east city limits. One hundred and sixty-four stations were located on these three cordons at points used by the Rapid Transit Commission in their study of vehicular traffic.

Traffic counts were taken in the seven cities during the month of August, 1931. Vehicles were classified as local, nonlocal, or non-Michigan. Local traffic was identified through the registration numbers assigned for cars of each city by the State Motor Vehicle Director. The correctness of such classification was checked by noting the license numbers and determining the domicile of the owner, in order to eliminate from the local classification those vehicles whose owners bought tags in the city but who were not domiciled within the city limits.

Where the volume of traffic was not too heavy all cars were classified and the domicile of the owner determined. Where this could not be done, as many license numbers as possible were noted and later classified. About one in each three license numbers was taken for investigation upon each route at a station.

A certain percentage of apparent local traffic (i. e., vehicles carrying tag numbers assigned to the city of observation) was found to be of nonlocal origin. This percentage was applied to each original count of vehicles and the traffic of true local origin determined. For example, at station 27 in Grand Rapids 1,551 cars with apparent local tags were noted on the north route at the intersection of Ionia Avenue and Crescent Street on Monday, August 24, during the hours from 10 a. m. to 8 p. m. Examination disclosed that 137, or 8.8 per cent of these cars were owned outside of Grand Rapids. The vehicle count on this route totaled 3,492 apparent local cars. Subtracting 307 or 8.8 per cent of the count gave 3,185 local vehicles. The 307 so deducted were added to the nonlocal classification.

Population and type of traffic.—The ratio of local traffic to total traffic ordinarily increases with population. In Figure 2

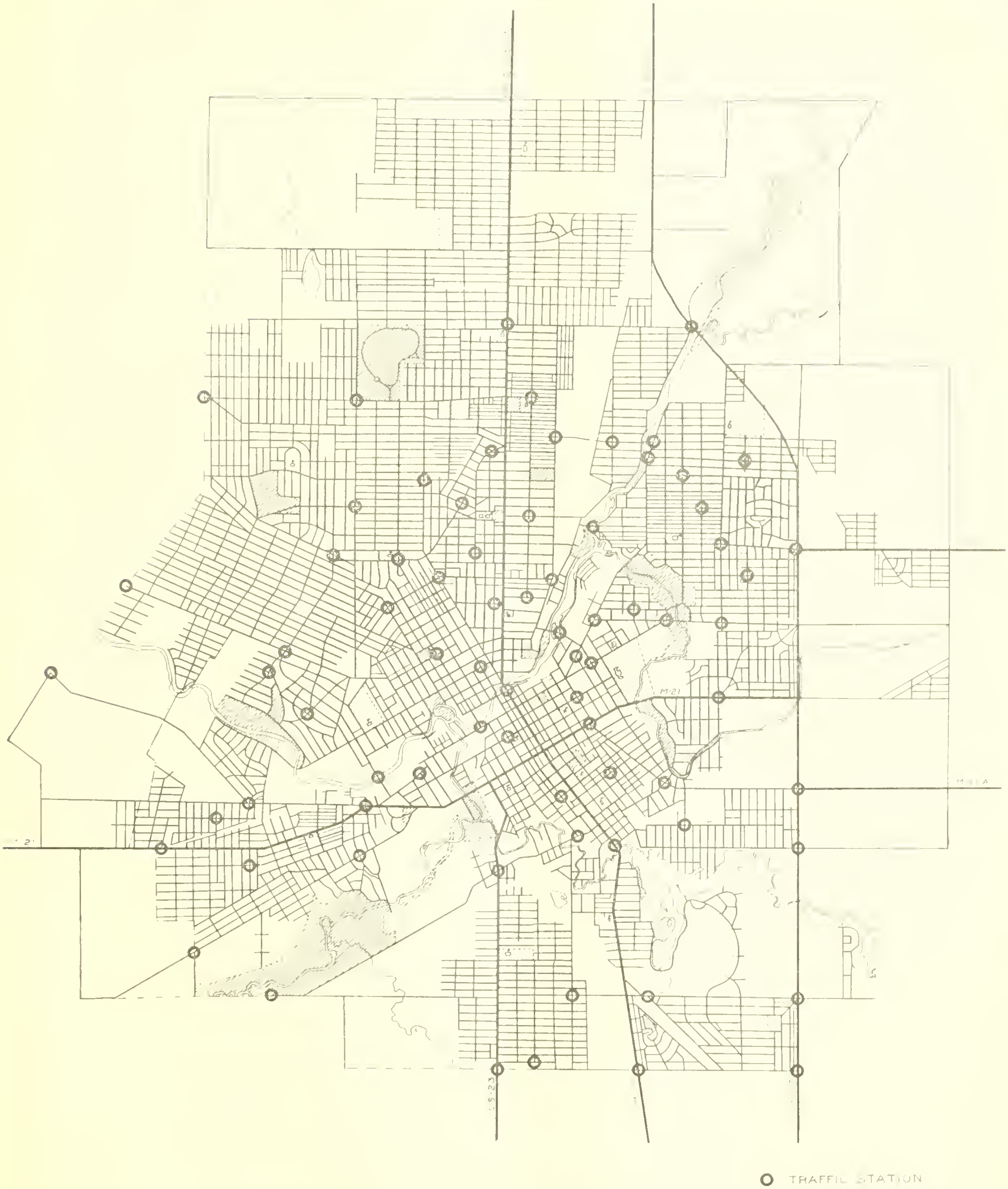


FIGURE 1.—MAP SHOWING LOCATIONS OF TRAFFIC STATIONS IN CITY OF FLINT, MICH.

the percentage of local traffic (determined by weighting percentages on through and nonthrough streets by mileage in each class) is plotted against population and a trend line drawn. With the exception of Jackson and Lansing, it is apparent that the larger the city the greater the percentage of local traffic. After a population of 180,000 is reached, the effect of increases in population upon the percentage of local traffic is very small. Detroit with nearly nine times the population of Grand Rapids, had but a slightly higher percentage of local traffic, although the volume of Detroit traffic is several times that of Grand Rapids.

Through street mileage and type of traffic. Since the data for Jackson and Lansing do not agree with the population-local traffic trend, and because there are other cities of about this size for which traffic data were not taken, another method was developed as a check and a modifier of the results obtained by considering population and percentage of local traffic. This was

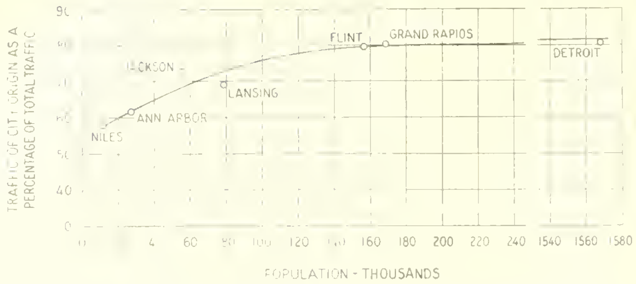


Figure 2.—Relationship Between Local Traffic on City Streets and Population of Cities; Percentage of Local Traffic Determined by Weighting Percentages on Through and Nonthrough Streets

done by plotting the ratio of through street mileage to total mileage for each city against the percentage of local traffic observed in each of the seven cities. There is an inverse relationship between these two factors as shown in Figure 3.

With a trend line through the points thus obtained, it is possible to determine the percentage of local traffic for any city if its ratio of through street mileage to total mileage is known, thus enabling close estimates of percentage of local traffic in those cities where no traffic data were obtained.

The average percentage of local traffic for a number of cities combined, when derived by both methods, varies only slightly. From consideration of data from the seven cities studied it appears that the relationship of percentage of through street mileage to local traffic provides an excellent check upon the population-local traffic relationship, and that it may be a better measure of local traffic in the individual city.

Although Jackson and Lansing diverge from the population-local traffic trend in the percentage of through street mileage-local traffic relationship, they are on or very near the trend line, but in reverse order. In Figure 3 Grand Rapids and Detroit are farthest from the trend line and on opposite sides. Grand Rapids has an unusual number of parallel through streets. More Streets in Detroit might properly be included in the through-street mileage (thus bringing this city closer to the trend line), but were not so included in conformance with the definition of through routes. There is also the fact that in Detroit through streets are much wider than local streets, so that the area of through-street mileage and that of local-street mileage is more nearly in proportion to that of the other cities.

Local traffic in Detroit and adjacent cities.—As previously noted, the traffic stations in Detroit were located on three cordons. These cordons were located at about 3, 6, and 8 miles from the center of the city.

Table 1 gives detailed figures for local traffic on each cordon as well as for each section of the cordons. The percentage of local traffic at each cordon was as follows: Inner, 81.2; middle, 81; and outer, 65.8. The observed local traffic at all cordons combined was 79.9 per cent.

Table 1.—*Local traffic at Detroit cordons*

Section	Inner cordon		Middle cordon		Outer cordon	
	North and South	East and West	North and South	East and West	North and South	East and West
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
East side	81.0	84.8	83.3	84.2	77.7	78.1
North side	83.0	81.0	77.6	81.9	61.7	64.6
West side	79.2	80.0	82.3	78.2	--------	58.8
Total	81.2		81.0		65.8	

The traffic of cities within and surrounding Detroit differs from that of an average city of similar size located at a distance from a large city. Hamtramck and Highland Park are within the corporate limits, but are not a part of Detroit. Local traffic in these places is similar to that of Detroit considering the entire area as a unit. Several of the Detroit traffic stations on the middle cordon were located in Highland Park. The counts at these stations showed that Detroit accounted for 77.4 per cent of the traffic, and 2.6 per cent originated outside of the State. Only 20 per cent was originated in Highland Park and the rest of Michigan outside of Detroit.

The population-local traffic relationship indicates a percentage of local traffic for Highland Park of less than two-thirds that of Detroit, while the percentage of through street mileage-local traffic relationship indicates a percentage of local traffic a little less than that of Detroit. Either method indicates a figure several times the observed local traffic. It is only logical to expect local traffic in Hamtramck and Highland Park to be very much like, if not identical with, that of Detroit if all three were combined as a metropolitan area.

Combining all these suburban places into one area and treating them as a Detroit metropolitan area brings the whole in nearer agreement to the trend line of Figure 2.

Method of estimating local traffic in all cities of State.—In arriving at estimates of the average percentage of local traffic in Michigan cities the two methods previously explained were used. The percentage of local traffic in the cities of the survey, when determined by population ratios, by ratio of through street mileage to total mileage, and by actual count, is not always the same, as will be seen by referring to Figures 2 and 3. Location of a town or city on heavily traveled through routes near a large city, or some other determining factor for the particular city, may be of great importance in arriving at the correct percentage of local traffic for a city. The heavy traffic on through routes passing through a small town lowers the ratio of local traffic to total traffic to a very low figure as compared with that of a town of the same size not on a through route, or on a through route with a much lower volume of traffic.

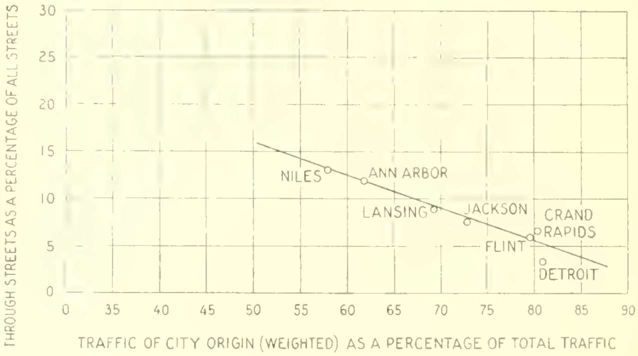


Figure 3.—Relationship Between Local Traffic on City Streets and Mileage of Through Streets; Percentage of Local Traffic Determined by Weighting Percentages on Through and Nonthrough Streets by Mileage in Each Class

Since no towns or cities under 10,000 population were included in the cities surveyed, an estimate of local and nonlocal traffic for these can be made on the basis of the trend line of Figure 2 extended or the ratio of through street mileage to total mileage as scaled from maps.

However, in the application of the through-street percentage relationship to small towns, it seems reasonable to include county routes which enter the town as connections with State or Federal-aid routes in the through-street mileage. It appears that county routes leading into small towns have about the same effect upon local traffic as have State routes entering larger cities. In fact, some towns have no State routes passing through them, so it is necessary to use county routes.

Whatever the method used to determine the percentage of local traffic in places under 10,000 population, it will have little effect on the mean local traffic for all Michigan cities combined. Including all incorporated places under 10,000 population, however small, the average percentage of local traffic for these would have to vary more than 10 per cent to affect the mean local traffic of all Michigan cities combined by as much as 2 per cent.

If all incorporated places under 10,000 population had been included in the city survey, it is doubtful if the combined average would vary materially from an average arrived at by use of either the population or through-street percentage relationships to local traffic.

Local traffic at single stations on through routes varies from 8 to 54 per cent for towns and cities of varying sizes under 10,000 population. One town under 1,000 population had 14 per cent local traffic on two intersecting streets combined but on one of these streets the local traffic was 43 per cent. The other street was a heavily traveled through route which greatly reduced the average for both streets.

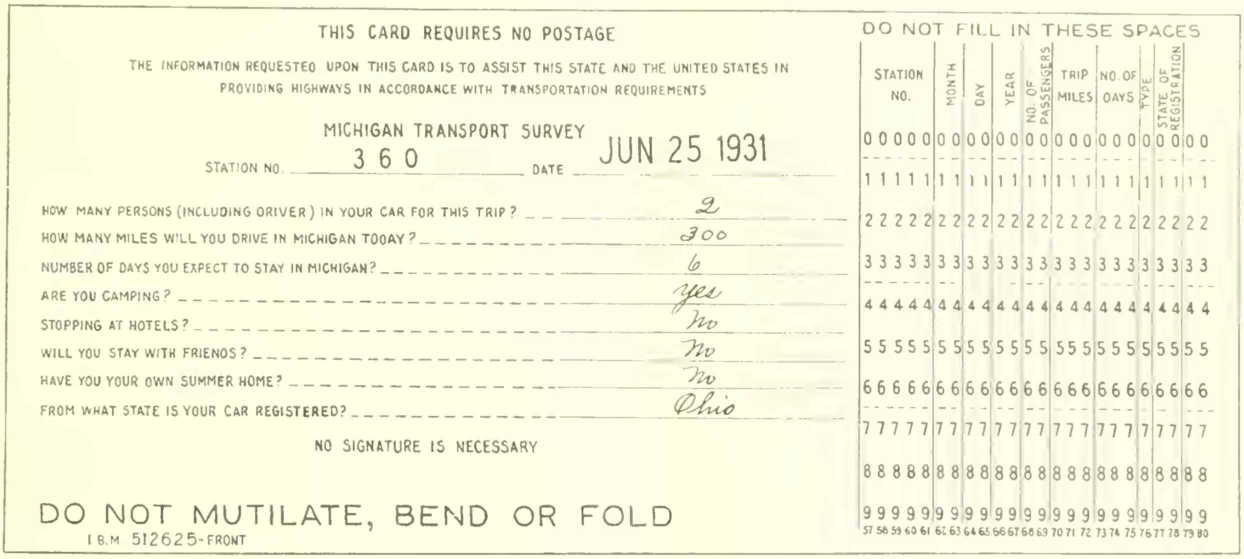
In estimating the percentage of local traffic in towns and cities under 10,000 population, all of these cities were considered as an average city within the group. Also, sample towns were con-

(Continued on p. 200)

MOTOR TOURIST TRAFFIC IN MICHIGAN

By the Bureau of Public Roads, United States Department of Agriculture, and the Michigan Highway Department

THE following analysis of motor tourist traffic is based upon post-card questionnaires distributed during the course of a traffic survey extending from July, 1930, to July, 1931. A sample card is shown in Figure 1. More than 42,000 of these cards were returned, and all sections of the State are represented.



THIS CARD REQUIRES NO POSTAGE

THE INFORMATION REQUESTED UPON THIS CARD IS TO ASSIST THIS STATE AND THE UNITED STATES IN PROVIDING HIGHWAYS IN ACCORDANCE WITH TRANSPORTATION REQUIREMENTS

MICHIGAN TRANSPORT SURVEY

STATION NO. 3 6 0 DATE JUN 25 1931

HOW MANY PERSONS (INCLUDING DRIVER) IN YOUR CAR FOR THIS TRIP? 2

HOW MANY MILES WILL YOU DRIVE IN MICHIGAN TODAY? 300

NUMBER OF DAYS YOU EXPECT TO STAY IN MICHIGAN? 6

ARE YOU CAMPING? yes

STOPPING AT HOTELS? No

WILL YOU STAY WITH FRIENDS? No

HAVE YOU YOUR OWN SUMMER HOME? No

FROM WHAT STATE IS YOUR CAR REGISTERED? Ohio

NO SIGNATURE IS NECESSARY

DO NOT MUTILATE, BEND OR FOLD

I.B.M. 512625-FRONT

DO NOT FILL IN THESE SPACES

STATION NO.	MONTH	DAY	YEAR	NO. OF PASSENGERS	TRIP MILES	NO. OF DAYS	TYPE	STATE OF REGISTRATION	
0 0 0 0 0	0 0	0 0	0 0	0 0	0 0 0	0 0 0	0	0 0	0 0
1 1 1 1 1	1 1	1 1	1 1	1 1	1 1 1	1 1 1	1	1 1	1 1
2 2 2 2 2	2 2	2 2	2 2	2 2	2 2 2	2 2 2	2	2 2	2 2
3 3 3 3 3	3 3	3 3	3 3	3 3	3 3 3	3 3 3	3	3 3	3 3
4 4 4 4 4	4 4	4 4	4 4	4 4	4 4 4	4 4 4	4	4 4	4 4
5 5 5 5 5	5 5	5 5	5 5	5 5	5 5 5	5 5 5	5	5 5	5 5
6 6 6 6 6	6 6	6 6	6 6	6 6	6 6 6	6 6 6	6	6 6	6 6
7 7 7 7 7	7 7	7 7	7 7	7 7	7 7 7	7 7 7	7	7 7	7 7
8 8 8 8 8	8 8	8 8	8 8	8 8	8 8 8	8 8 8	8	8 8	8 8
9 9 9 9 9	9 9	9 9	9 9	9 9	9 9 9	9 9 9	9	9 9	9 9
57 58 59 60 61	62 63	64 65	66 67	68 69	70 71 72	73 74 75	76	77 78	79 80

FIGURE 1.—CARD DISTRIBUTED TO TOURISTS IN MICHIGAN

All operators of foreign cars were given cards at stations near the border but since stations were not operated continuously the operators stopped at stations in the interior of the State and found to be without such a card were given one. The information upon these cards, supplemented by special data obtained at the Ambassador Bridge, the Detroit-Windsor Tunnel, and all ferries, furnished data with regard to tourist and Canadian traffic movements.

The origin of tourist cars is summarized in Table 1, and also in Figure 2, in which the percentage of tourist traffic originating in the areas indicated is proportional to the area of the circle in each case. Most of this traffic originates in the States adjoining Michigan and in Canada, this area contributing 82.1 per cent of the total tourist traffic. The remaining 18 per cent originates in all parts of the United States, the bulk of it coming from the northeastern States and from the central plains States. Of the traffic from the central plains States, more than one-third originates in Minnesota.

TABLE 1.—*Origin of tourist traffic in Michigan during 1930 31, as indicated by questionnaire cards*

State, country, or group	Number of cards returned	Percentage of total
Wisconsin	2,528	6.0
Illinois	13,608	32.5
Indiana	8,115	19.4
Ohio	8,942	21.4
Canada	1,168	2.8
Total, neighboring States and Canada	34,361	82.1
New England States	410	1.0
Northeastern States	2,382	5.7
Southeastern States	992	2.4
Central States	2,823	6.7
Western States	901	2.1
Total	41,869	100.0

A classification of tourist cars by type of accommodation, Table 2, indicates that more than one-third of the total number of visitors stayed with friends, nearly 24 per cent at hotels, and more than 19 per cent at summer homes. These three important groups comprise four-fifths of the total. More than half of the remainder proceeded directly through the State, making no overnight stops. Camping parties represent only 6 per cent of the total, and the miscellaneous group, those cars which could not be conveniently classified under any of the above types, 4 per cent.

The average number of persons per car in all tourist cars was 2.8. More than one-half of the total cars carried 1 or 2 persons and nearly 70 per cent carried 1, 2, or 3 persons. Every sixth car carried four persons but the number of cars carrying five or more persons was only a small part of the total.

TABLE 2.—*Distribution of tourist cars by type of accommodation*

Type	Number of cars	Number of cars as percentage of total	Number of persons	Number of persons as percentage of total
Friends	14,094	33.53	43,638	36.84
Hotels	12,348	29.38	27,999	23.64
Summer home [1]	7,622	18.13	23,232	19.61
Through traffic	4,021	9.57	11,793	9.96
Camping [2]	2,236	5.32	7,095	5.99
Miscellaneous [3]	1,712	4.07	4,694	3.96
Total	42,033	100.00	118,451	100.00

[1] "Summer home" includes all cars where possession of a summer home was indicated, on the assumption that other types of accommodation were used only incidentally in going to and from the summer home.

[2] Includes camping and friends.

[3] Miscellaneous includes cars unclassified according to type, and the minor groups: Hotels and camps; hotels and friends, and, hotels, camps, and friends.

The average length of stay for all classes of tourists is 11 days per party. It ranges from one day, reported by one car in every five, to more than six months. The greatest number of visits are of short duration. Nearly 40 per cent of all parties stayed two days or less, while visits of one week or less were made by nearly three-fourths of all parties. Table 3 gives the length of stay of tourist cars, in percentages distributed by type of accommodation used.

There is naturally a considerable difference in the average length of stay of the various types of tourists, ranging from 7.5 days for those staying with friends, to 24 days for those using their own summer homes. The average stay for those staying at hotels is 8.2 days, and that for campers is 10.6 days.

Table 3.—*Length of stay of tourist cars in percentages distributed by type of accommodation used*

Number of days	Friends	Hotels	Summer home	Camping	Miscellaneous	Total
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	19.8	17.2	16.5	12.5	6.8	21.0
2	21.2	20.7	15.0	15.7	12.5	18.2
3	15.1	13.3	9.9	11.8	10.9	12.5
4	9.0	8.7	5.3	7.2	9.7	7.8
5	5.3	7.4	3.5	5.1	7.3	5.4
6	2.9	3.6	2.0	4.5	4.9	3.0
7	6.1	5.0	4.2	7.6	7.6	5.3
8	1.3	1.3	.9	2.7	2.9	1.3
9	.3	.4	.4	.8	1.0	.4
10	4.8	6.1	4.4	7.6	7.8	5.2
11	.2	.2	.1	.4	.6	.2
12	.7	.7	.8	1.6	1.6	.8
13	.1	.1	.1	.2	.1	.1
14	3.9	3.9	3.7	6.7	6.7	4.0
15	1.0	1.3	1.1	1.8	2.1	1.2
16	.2	.2	.3	.4	.3	.2
17	----------	.1	.2	----------	.1	.1
18	.2	.4	.3	.6	.2	.3
19	----------	----------	----------	.1	.1	----------
20	.6	.7	.7	.8	1.4	.7
21 to 30	1.4	2.4	2.6	2.7	2.8	2.0
STAY IN MONTHS						
1 to 2	3.4	3.9	6.9	5.1	6.3	4.3
2 to 3	1.3	1.3	10.4	2.7	3.2	3.1
3 to 4	.9	.8	8.3	1.1	1.6	2.2
4 to 5	.1	.1	1.4	.2	.1	.3
5 to 6	----------	----------	.4	----------	.2	.1
6 to 12	.2	.2	.6	.1	1.2	.3
Total	100.0	100.0	100.0	100.0	100.0	100.0
Average (days)	7.5	8.2	24.0	10.6	14.7	11.0
Median (days)	3.6	3.9	4.5	5.5	6.6	3.9

ESTIMATE OF TOURIST EXPENDITURES

The information concerning length of stay, type of accommodation, and average number of passengers may be applied to the total incoming tourist traffic to estimate the value of this traffic in terms of expenditures within the State. However, the accuracy of such an estimate depends primarily on the estimate of the average expenditure per tourist per day, and very little accurate data[1] has been collected on this point. The final estimate here given is regarded merely as an indication of total tourist expenditures, but conservative figures have been used throughout and it is, therefore, probable that the actual expenditure is greater than the estimate.

As the cost of car operation is an item of expense for all types of motor tourists, it is convenient to assume an average cost per mile and apply this figure in the estimate for each type. The following estimate of car-operation costs is based upon the assumption that the average tourist car is a light 6-cylinder model. Cars of this type were used by recorders and supervisors in the survey, and the following unit costs are, therefore, taken directly from the cost records of these cars:

Average miles per gallon of gasoline	17
Average miles per quart of oil	100
Average cost per gallon of gasoline	$0.17
Average cost per quart of oil	.27
Cost of gasoline per mile	.010
Cost of oil per mile	.003
Miscellaneous costs per mile	.005
Total cost of operation per mile	.018

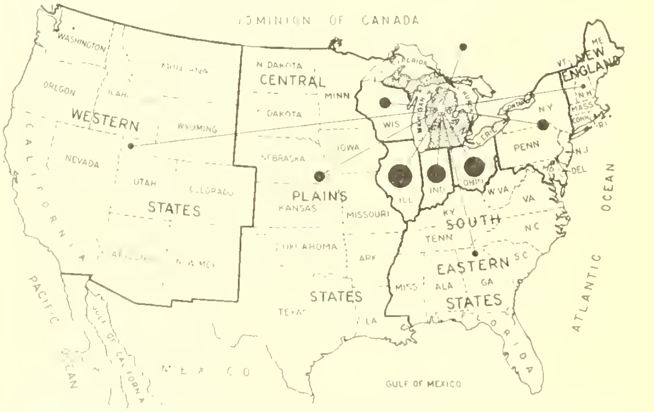


Figure 2.—Origins of Motor Tourist Traffic in Michigan. Size of Circle Indicates Relative Number of Tourists from Each Area

This figure of $0.02 per mile may appear to be too conservative, but it should be remembered that it represents only direct operation expenditures which would be made in Michigan during a visit. Indirect costs such as depreciation, license fees, etc., are not considered.

Table 4 is an itemized estimate of the daily expenditures per person for the six tourist types. The estimate for the summer-home group is based upon an assumed average home value of $3,000, upon which carrying charges and all operating costs on a yearly basis will approximate 10 per cent of the value, or $300. This expenditure is charged against a season of 100 days, resulting in the equivalent of a rental value of $3 per day, or $1 per day per person for a party of three.

Table 4.—*Itemized estimate of expenditures per person per day for various types of tourists*

Items	Friends	Hotels	Summer home	Camping	Through	Miscellaneous
Average number of persons per party [1]	3.1	2.3	3.1	3.2	2.9	-----
Average daily miles per car	50	100	50	100	[2] 120	-----
Cost of car operation at $0.02 per mile	$1.00	$2.00	$1.00	$2.00	$2.40	-----
Cost of garaging	-----	.75	-----	-----	-----	-----
Total car operation cost per day	1.00	2.75	1.00	2.00	2.40	-----
Cost of car operation per person per day	.32	1.20	.32	.62	.83	-----
Cost of lodging per person per day	-----	[3] 2.70	[4] 1.00	.75	-----	-----
Cost of food per person per day	.75	2.25	.75	1.00	1.50	-----
Miscellaneous costs per person per day	.75	1.50	.75	1.00	.50	-----
Total expenditures per person per day	1.82	7.65	2.82	3.37	2.83	[5] 3.70

[1] Averages obtained from questionnaires.

[2] Average obtained from questionnaires. All other daily mileages are estimates, as the averages obtained from the questionnaires are not considered representative averages for daily mileage for the entire length of stay.

[3] Based on an assumed cost of $4 per day for one person; $5 per day for two persons; and reduced to a unit cost for a party of 2.3 persons.

[4] Based on an average rental value of $3 per day for a period of 100 days.

[5] Arithmetic average of the expenditures for all other types.

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[1] Most of the estimates of the value of tourist traffic have been made by tourist associations and automobile clubs. In the majority of cases, the estimates are based frankly upon guesses as to the average expenditure per person per day. Several studies of considerable merit have been made by means of questionnaires and wherever possible the following estimates of expenditures by the various types of tourists have been checked against these studies.

The average daily mileages per car given in Table 4 were estimated, except in the case of the cars passing through the State, for which the average obtained from the questionnaires was used. The questionnaire averages in general refer to daily mileages while touring, and are not representative of the daily travel of parties making an extended stay in the State. The estimates used represent an attempt to allow for both the trip mileage in and out of the State, and the daily use of the vehicle during the sojourn.

The average daily volume of foreign cars at selected border stations was used in computing the total yearly volume of tourist traffic. A considerable portion of foreign traffic near the border is an "over-the-line" movement of cars for business purposes, and it was necessary to apply an appropriate correction at each station to eliminate this traffic. The proportion of cars staying one day or less from the State immediately adjacent to each station as obtained from the questionnaire cards was considered a reliable indication of this business traffic. These proportions were accordingly applied to the total foreign traffic at each station under consideration and the remainder was assumed to represent the true volume of tourist traffic, shown in column six of Table 5. The total daily volume of tourist traffic at all border stations was 13,700 cars per day and represents both incoming and outgoing cars. One-half of the above total, or 6,850 cars per day, may be considered as incoming tourist cars, or 2,500,000 cars per year.

TABLE 5.—*Estimate of automobile tourist traffic per year*

Station	Route	Direction	Daily foreign passenger cars	Reduction factor	Net daily foreign passenger cars
1	Ferry		869	0.96	845
2	do		657	.87	571
3	do		711	.86	611
4	do		195	.90	176
5	do		6	1.00	6
6	do		4	1.00	4
7	do		234	.78	182
8	do		94	.85	80
249	US 12	N. and S.	2,546	.71	1,808
250	CR	S.	121	.67	81
251	US 31	N. and S.	1,976	.67	1,324
260	US 27	S.	398	.81	326
280	US 112	S.	851	.76	647
320	US 2	W.	173	.93	161
321	US 141	SE.	291	.84	244
351	US 2	E. and W.	197	.94	185
353	M 62	W.	62	.89	55
355	M 26	N. and S.	82	.87	71
372	M 34	S.	58	.82	48
410	M 73	SW.	45	.92	41
412	US 2	S.	76	1.00	76
468	M 34	W.	53	.91	48
-----	CR	S.	36	.91	33
474	M 52	S.	155	.59	91
519	CR	W.	13	.92	12
521	US 41	S.	154	.87	134
522	M 35	S.	108	1.00	108
538	US 223	N.	1,127	.60	676
539	US 23	N.	1,086	.81	880
540	US 24	N. and S.	2,312	.88	2,034
541	US 25	N. and S.	1,795	.88	1,580
653	TR	S.	20	.82	16
654	US 131	S.	593	.85	504
655	CR	S.	7	.82	6
838	CR	W.	9	1.00	9
944	CR	S.	28	1.00	28
	Total per day				13,701
	Incoming per day				6,850
	Incoming per year				2,500,250

TABLE 6. *Estimated yearly tourist expenditures by types*

Type	Person-days per car [1]	Expenditures per person-day	Expenditures per car per visit	Cars entering per year	Expenditures per year
Friends	22.1	$1.82	$40.22	837,500	$33,700,000
Hotels	17.5	7.65	133.88	735,000	98,400,000
Summer home	77.6	2.82	218.83	452,500	99,000,000
Camping	31.6	3.37	106.49	240,000	25,600,000
Through	3.0	2.83	8.49	132,500	1,100,000
Miscellaneous	43.0	3.70	159.10	102,500	16,300,000
	31.6	3.47	109.64	2,500,000	274,100,000

[1] Since unit expenditures were estimated on a person-per-day basis, length of stay is expressed in the unit "person-days" for purposes of computation. It does not correspond exactly to the figure obtained by multiplying the average number of persons per car by the average stay per car because the average number of persons per car varies by length of stay as well as by type.

In Table 6 the estimated expenditures of the various types of tourists and the percentage of each type are applied to the total incoming traffic to obtain the total expenditures for each type. The total expenditures of all motor tourists were approximately $274,000,000. While this estimate is admittedly an approximation, it serves to impress the tremendous value of this traffic to the State. Total foreign vehicle-mileage indicates a consumption by foreign cars of more than 30,000,000 gallons of gasoline, and gasoline taxes paid by owners of foreign vehicles amounts to more than $900,000 annually.

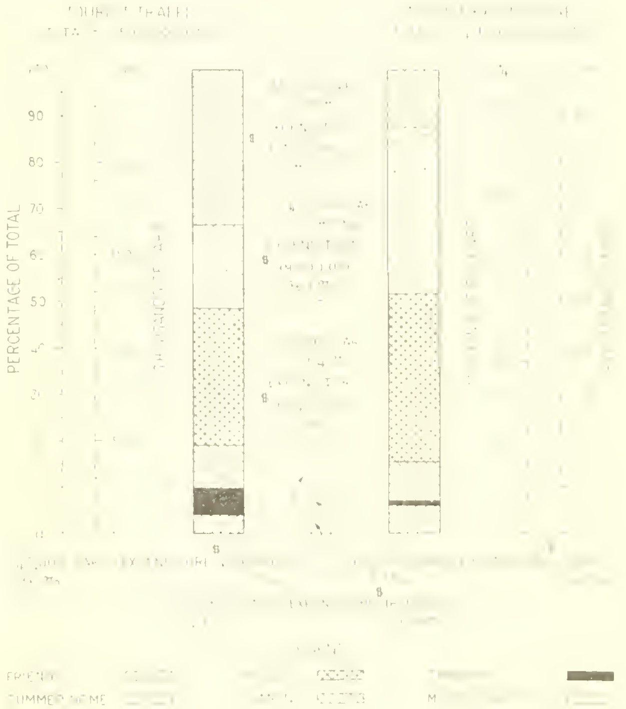

FIGURE 3.—NUMBER OF TOURIST CARS AND ESTIMATE OF EXPENDITURES BY CLASS OF TOURIST

Of equal if not greater importance than the total expenditures of motor tourists is the relation between the volume of each type of tourist traffic and the expenditures of that group. Tourists owning their own summer homes, representing only 18.1 per cent of the total tourist traffic, are responsible for 36.1 per cent of the total tourist expenditures, more than any other group. Furthermore, the expenditures per visit of this class are considerably greater than that of any other, and the advantage of endeavoring to increase the number of this kind of tourists is apparent. Only slightly less valuable is the hotel visitor. The group using hotels forms 29.4 per cent of the total traffic, but spends $98,400,000 per year, nearly 36 per cent of the total. These two groups, comprising slightly less than one-half of the traffic spend nearly three-fourths of the total expenditures.

These relationships between volume of traffic and expenditures for the various types of tourists are shown in Figure 3.

There is an important interchange of traffic along the Canadian border, but what might be termed the "balance of tourist trade" is heavily in favor of Canada, considerably more cars bearing Michigan tags or those of other States entering Canada than enter Michigan with Canadian tags. At eight ferries and bridges there was an average passenger car volume of 9,696 per day. Michigan cars constituted 71.4 per cent of the total, cars from other States, 9.0 per cent, and Canadian cars, 19.6 per cent, as shown in Table 7.

The greatest volume of this traffic naturally occurs between Detroit and Windsor, the Ambassador Bridge carrying an average of 3,255 cars per day; the Bates Street Tunnel, 2,141; the Windsor Ferry, 2,504; and the Walkerville Ferry, 986. The largest part of this traffic is probably an exchange of business traffic between Detroit and Windsor, but there is a very considerable volume of tourist traffic, as indicated by the volume of traffic at these points from other States. Detroit is a convenient point of entry into Canada for tourists from the United States because it is the terminus of main highways from Chicago and Toledo, while Windsor is the terminus of a popular route from Niagara Falls.

TABLE 7.—*Density of traffic at bridges and ferries connecting with Canada*

Location	Average daily passenger cars							Average daily passenger cars and trucks
	Michigan cars		Foreign cars				Total	
			Canadian cars		Other cars			
	Number	*Per cent*	*Number*	*Per cent*	*Number*	*Per cent*		
Ambassador Bridge	2,386	73.3	449	13.8	420	12.9	3,255	3,401
Bates Street Tunnel	1,484	69.3	497	23.2	160	7.5	2,141	2,164
Windsor Ferry	1,793	71.6	573	22.9	138	5.5	2,504	2,581
Walkerville Ferry	791	80.2	164	16.6	31	3.2	986	1,101
Port Lambton Ferry	27	81.8	5	15.2	1	3.0	33	34
St. Clair Ferry	17	85.0	3	15.0	(1)	--	20	22
Port Huron Ferry	354	60.2	158	26.9	76	12.9	588	592
Sault Ste. Marie Ferry	75	44.4	48	28.4	46	27.2	169	177
Total and percentage	6,927	71.4	1,897	19.6	872	9.0	9,696	10,072

[1] Less than one car per day.

Traffic of Canadian cars between Detroit and Windsor aggregates 1,683 per day, nearly one-fifth of the total, and card returns indicate that about 50 per cent of this volume is tourist traffic.

Traffic at other points along the Canadian border is very much lighter than at Detroit, although there is a considerable volume at Port Huron and Sault Ste. Marie. The Port Huron Ferry carries an average of 588 passenger cars per day—60.2 per cent being Michigan cars, 12.9 per cent from other States, and 26.9 per cent from Canada. The ferry at Sault Ste. Marie carries an average daily traffic of 169 cars—44.4 per cent of which are from Michigan, 27.2 per cent from other States, and 28.4 per cent from Canada. The ferries at Port Lambton and St. Clair are not on main routes, and, in consequence, carry a relatively small volume of traffic.

(Continued from p. 196)

sidered in connection with the through-street-mileage relationship. Studying all available data the typical city for the group under 10,000 population was estimated to have 43 per cent local traffic.

Table 2 presents the data on city traffic of local origin for all cities in Michigan. Detroit is listed separately in the table because of its size and effect upon any average. The other six sample cities are listed next and in combination with Detroit. Then follows the data for group 1 and group 2 cities. Group 1 cities are those for which street mileage data, as well as population, are available. Group 2 cities are those with more than 10,000 population, but for which the mileage data are lacking. These are followed by the large group of towns and cities under 10,000 population.

The procedure used in combining the results in the Michigan cities was as follows: (1) Field data for the seven sample cities were summarized; (2) percentage of local traffic determined by the population-local traffic relationship; (3) percentage of local traffic determined by the use of through street percentage-local traffic relationship; (4) a final average secured by a combination of the results of 2 and 1, and 3 and 1, using the weighted average for each.

TABLE 2.—*Percentage of traffic of local origin in all Michigan cities*

City group	Population	Weighted average percentage of local traffic, as determined by—	
		Population of city	Mileage of through streets
		Per cent	*Per cent*
Detroit [1]	1,569,000	80.9	80.9
Six other cities of survey [1]	496,000	76.1	76.1
Seven cities of survey combined [1]	2,065,000	79.7	79.7
Cities over 10,000, group 1	572,000	64.8	65.6
Cities over 10,000, group 2	296,000	64.1	[2] 64.5
All cities over 10,000 combined	2,933,000	75.2	75.4
Cities under 10.000 [3]	663,000	43.0	43.0
All Michigan cities	3,596,000	69.3	69.5

[1] Actual percentages, from field data.
[2] Estimated from group 1 cities.
[3] Estimate determined by use of both population and through street mileage.

The percentages of local traffic in the third column of Table 2 are either the weighted averages obtained from field observations, or are obtained from the data of Figure 2. These percentages are weighted by the population of the city or group of cities to give an average of 69.3 per cent local traffic. The percentages of the last column are those obtained from the field data or from the data of Figure 3 and are again weighted by population to give a combined figure of 69.5 per cent local traffic in the cities of Michigan as a whole.

The differences in the final percentages obtained are slight, the figures varying from 69.3 to 69.5 per cent local traffic for the State as a whole. Since the weighted average for traffic of city origin probably best represents the actual condition, and since the unweighted averages vary little from the above figures, the final percentage of local traffic for all incorporated places in Michigan is estimated to be 69 per cent of the total traffic of these cities.

CORRECTION

Vol. 13, No. 10, December, 1932.—In the article entitled "The Problem of Motor Vehicle Regulation," Appendix B, page 168, the following note was included under the heading "Remarks," and appeared opposite the entries for the State of Maine:

> Truck tractor, 4-wheel semi and 6-wheel full trailer, maximum gross 80,000 pounds. No combination including more than 1 semitrailer or full trailer may be operated at more than 10 miles per hour.

The note should be read to apply to the State of Maryland.

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U S GOVERNMENT PRINTING OFFICE 1933

Clemson University
3 1604 011 534 254

FEDERAL
PUBLICATION

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DEMCO, INC. 38-2931